# NODE v.1.0 beta: attempting to prioritize large-scale seismic risk of engineering structures on the basis of *nominal deficit*

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ABSTRACT
Prioritization at a regional scale requires poor-input methodologies able to provide an, even rough and/or conventional, assessment of the seismic risk. The work in the present paper elaborates ideas from literature in an attempt to define relative (within the population under investigation), yet quantitative and structure-specific seismic risk indexes based on the comparison of code requirements at the time of design and current seismic demand. Despite it is virtually applicable only to engineering structures and has several limitations and strong assumptions, it also has the advantage of requiring extremely poor and easy to find data, such as year of design and location, and can be refined if further information is available. Moreover, being quantitative, it can take into account for both hazard and exposure, the latter in terms of individual or societal risk.
In the paper, the main assumptions and simplifications of the methodology are critically discussed, regarding the definition of inelastic demand, the inclusion in the assessment of local geology, and the treatment of buildings located in places classified as non-seismic when built. Finally, a prototype software tool incorporating the seismic classification and code evolution of the Italian territory from 1909 and able to compute structure-specific design base shear for any structure, is presented.

## 1 INTRODUCTION

Italy is usually considered to be a country with a moderate seismicity, yet high seismic risk. This is believed to be related to buildings' average lifespan, quality, lack in maintenance, and the fact that a large portion of the building stock (a significant fraction of which comprises non-engineering masonry structures) was designed without seismic provisions or in compliance with obsolete codes, with underestimation of the seismic actions. Therefore, in order to address resources to reduce it, the need for tools able to provide a seismic risk assessment on a regional, or even national, scale arises.

In this case, an explicit assessment of vulnerability by means of mechanical methods, commonly used for individual buildings, is not generally feasible requiring, in terms of time and costs, an effort incompatible with the scale of the problem. Moreover, the level of detail in needed data is not suitable for a regional approach.

Several methods have been developed in the literature for the evaluation of seismic risk on regional scale. They can be generally classified into qualitative (or empirical) and quantitative (or mechanical) categories.

Those belonging to the former category are essentially based on the statistical analysis of post-earthquake damage surveys. For this reason, their accuracy is often affected by the availability of damage data for different building classes.

These methods may also employ expert judgment for the identification of pre-defined structural vulnerability indicators (e.g., CNR-GNDT, 1993).

Quantitative methods for seismic risk assessment are aimed at the predictive evaluation of the seismic vulnerability of a building stock through mechanical evaluation of vulnerability. These aim at taking a structure-specific modelling approach at a class level (e.g., Cosenza *et al.*, 2005 and Iervolino *et al.*, 2007).

Multilevel approaches for risk-assessment and mitigation strategies are also available in literature. These usually include a first screening phase, with the aim of selecting the portion of structures within the portfolio with larger risk, for which a more refined level of analysis (e.g., structure-specific) is required. For example, American (ATC, 1978) and New Zealand (NZSEE, 2003) regulatory codes include criteria for defining acceptable risk levels and patterns for the definition of priorities.

With specific reference to the Italian context, we recall (Di Pasquale *et al.*, 2001) and (Goretti and Di Pasquale, 2004) and, more recently, the work by Grant *et al.*, (2007) for the definition of priorities of intervention on Italian school buildings.

In fact, on the premises of Grant *et al.* (2005 and 2007), in this paper a methodology for the assessment of seismic risk at regional scale and prioritization of intervention, implemented in a prototypal software, is discussed. It is based on the definition of a *relative* yet *quantitative* measure of seismic risk, based on the so-called “nominal deficit”. This is defined as the gap between design seismic demand according to the current seismic code and the one at the time of construction, which therefore is implicitly assumed to be the *nominal capacity* of the structure. This index takes into account vulnerability and hazard and may be extended to include exposure in social or individual terms, or a combination of the two.

Despite strong assumptions and limitations described in the following, it seems it can provide a tool for prioritization of seismic risk at a very large scale, which may be the first step of a typical multilevel approach for risk assessment and mitigation, which might include subsequent steps to address vulnerability of the structures top ranking in the first phase.

In the paper, after a brief review of the evolution of design provisions and seismic classification in Italy, the approach is presented. Subsequently, assumptions and critical issues are discussed. Finally, the NODE v.1.0 (beta) software, is presented.

## 2 EVOLUTION OF CODES AND SEISMIC CLASSIFICATION IN ITALY

Seismic regulatory codes in Italy have undergone a relevant number of changes since the first national seismic provision was enforced, after the Messina earthquake of 1908. These changes were, in general, consequence of catastrophic seismic events. In the following a few fundamental steps are reviewed; the reader should refer to (Di Pasquale *et al.*, 1999a and 1999b) for a much more comprehensive and informative review.

The first seismic building code was the royal decree of 1909 (R.D., 1909), according to which, some limitations about buildings’ height and structural typologies were given. Even if no indication about the entity of horizontal design forces was given, an expert panel determined a reasonable value of the so-called “seismic coefficient”, that is the ratio between horizontal seismic forces and total building weight, equal to 8% (this coefficient, may be seen as a design acceleration, measured as fractions of g, imposed to the structure).

The first explicit instruction regarding the value of the horizontal seismic forces was introduced in 1915 (R.D.L., 1915), after the earthquake in the Abruzzo region. This code established the seismic force equal to 1/8 and 1/6 of storey weight, for the first level and for the others, respectively.

In 1927 (R.D.L., 1927) a second seismic category, with respect to the one defined in the previous code, was introduced. It corresponded to a lower level of seismic horizontal forces and less restrictive structural provisions.

In the following years (from 1935 to 1975) no major changes in seismic provision were approved, with the exception for the changes in live loads, and the *declassification*[1] (due mainly to political pressure of local administrations) of several municipalities.

Moreover, between 1935 and 1937, the horizontal seismic coefficient was reduced, for the second category, from 7% to 5%.

An important change in Italian seismic code was enforced in 1975 (D.M., 1975), through the introduction of a response spectrum. For the first time design could be carried out performing dynamic or static analysis, with an horizontal force obtained as a function of seismic zonation, soil type, structural system, building structural period, and seismic weight, as reported in Equation 1:

[1] Removal of requirements for seismic design.

$$F_h = C \cdot R \cdot \varepsilon \cdot \beta \cdot W \quad (1)$$

where $W$ is the total weight of the building, $\varepsilon$ takes into account for *soil compressibility* ($\varepsilon$ = 1.00-1.30), $\beta$ considers the possible presence of structural walls ($\beta$ = 1.00-1.20) and $C$ represents the site seismicity (a sort of hazard), as defined in Equation 2:

$$C = \frac{S-2}{100} \quad (2)$$

where $S$ depends on the seismic classification ($S$ equal 12 and 9 for first and second category, respectively).

The $R$ coefficient in Equation 1 defined the spectral shape. Its expression, substantially unchanged until 2003, was defined as follows:

$$\begin{cases} R = 1 & \text{for} \quad T \le 0.8s \\ R = 0.862 \cdot T^{-2/3} & \text{for} \quad T > 0.8s \end{cases} \quad (3)$$

where $T$ is the fundamental structural period.

Further evolutions of regulatory provisions regarded the introduction of a third seismic category in 1981 (D.M., 1981), characterized by a S coefficient in Equation 2 equal to 6, and of an *importance factor* in 1984 (D.M., 1984). In the same years important enlargements of seismic zonation were approved in a way that, at the end of 1984, approximately 50% of Italian territory was considered to deserve seismic design.

Up to 1996 the only possible design approach was the *admissible stress*; nevertheless, in 1996 code (D.M., 1996) limit state design was introduced, with an amplification of seismic horizontal action of 1.5. However, the admissible stress approach was still allowed, so that the previous prescription was largely disregarded in practice.

The explanatory document attached to the 1996 code (M.LL.PP., 1997) contained first indications in the direction of capacity design. In fact, prescriptions aimed at the improvement of local ductility, were given.

The 2003 seismic code (O.P.C.M. 3, 2003) and its following modifications (O.P.C.M., 2005) represented the most relevant change in Italian seismic provisions over the preceding thirty years. In fact, with these documents, Eurocode 8 (CEN, 2004) approach was implemented, and the whole Italian territory was considered to be seismic, through the definition of a fourth seismic category.

Moreover, thanks to the work of the *Istituto Nazionale di Geofisica e Vulcanologia* (INGV), design actions were defined, on a municipal basis, by the value of peak ground acceleration (PGA) on rock with an exceedance return period of 475 yr, probabilistically evaluated according seismic hazard analysis (Stucchi *et al.*, 2011).

The horizontal seismic force was defined as:

$$F_h = S_a(T) \cdot W \cdot \lambda / (q \cdot g) \quad (4)$$

where $\lambda$ is a coefficient equal to 0.85 for static analysis and 1 otherwise, $S_a(T)$ is the spectral elastic acceleration (determined on the basis of a standard spectral shape anchored to the mentioned PGA) at the fundamental period $T$, $q$ is the *behaviour factor* and $g$ is the gravity acceleration.

Despite its major changes, this code was compulsory only for *strategic* buildings[2], and the practitioner was still allowed to use the 1996 code for ordinary constructions.

The last regulatory (D.M., 2008; in the following indicated as new Italian building code or NIBC), enforced in 2009 after L'Aquila earthquake, considered seismic hazard defined as a function of geographic coordinates of the construction site, and no longer on municipality basis.

With this code, design spectra defined in NIBC, although given with standard functional form, practically coincide with *uniform hazard spectra* (at least on rock) for the considered site. The corresponding exceedance probability depends on the limit state of interest, the type, and the nominal life of the structure. In case of different soil classes, coefficients apply to amplify accordingly.

As an example, in Figure 1 evolution of code-based seismic actions trough the years for L'Aquila downtown site (Lat: 42.36, Lon: 13.41) are reported, in terms of seismic coefficient or response spectrum (when defined).

Note that, starting from 2003, the spectrum is elastic with 5% damping. The latter has to be reduced by a so-called "behaviour factor", accounting for inelastic behaviour.

[2] Strategic buildings are those which, in the case of a seismic event, assume civil protection functions or can have significant consequences in terms of losses (OPCM, 2003).

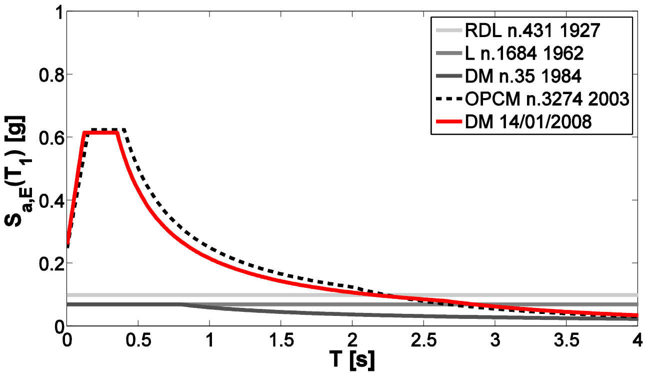


Figure 1. Seismic demand through years for L'Aquila site (Lat: 42.36, Lon: 13.41). Response spectra for 2003 and 2008 years are elastic and 5% damped.

Table 1, rearranged from Di Pasquale *et al.* (1999a and 1999b), summarizes the most relevant changes in base shear provisions and classification of Italian territory from 1909 to 2008. In it, the number of municipalities which changed classification is expressed with respect to the preceding code.

Finally, in Figure 2 the most important changes in the seismic classification of Italian territory are reported.

Table 1. Most important changes in horizontal actions and classification of territory, provided by Italian regulatory codes. (Di Pasquale *et al.* 1999b).

| Date | Horizontal seismic action | Changes in seismic classification |
|---|---|---|
| 18/04/1909 | Undefined | first zonation (367 municipalities) |
| 05/11/1916 | $F_h$ = *1/8 W* first floor<br>$F_h$ = *1/6 W* other floors | 416 municipalities classified |
| 13/03/1927 | **I cat:** $F_h$ = *1/8 W* first floor<br>$F_h$ = *1/6 W* other floors<br>**II cat:** $F_h$ = *1/10 W* | 2nd category; 951 municip. chnged zone |
| 13/03/1935 | **I cat:** $F_h$ = *0.10 W*<br>**II cat:** $F_h$ = *0.05 W* | 174 municipalities changed zone |
| 22/11/1937 | **I cat:** $F_h$ = *0.10 W*<br>**II cat:** $F_h$ = *0.07 W* | declassifications |
| 25/11/1962 | **I cat:** $F_h$ = *0.10 W*<br>**II cat:** $F_h$ = *0.07 W* | declassifications |
| 25/11/1975 | $F_h = C\,R\,\varepsilon\,\beta\,W$<br>**I cat:** *C = 0.10;*<br>**II cat:** *C = 0.07* | 239 municipalities change zone |
| 03/06/1981 | Unchanged, except for<br>**III cat:** *C = 0.04* | 3rd category; 239 municip. changed zone |
| 19/06/1984 | Unchanged except for<br>I = 1.4 strategic buildings<br>I = 1.2 crowded buildings | 1533 municipalities changed zone |
| 16/01/1996 | Limit State Design | - |
| 20/03/2003 | $F_h = S_d(T)\,W\,\lambda\,/\,(q\,g)$<br>**I zone:** $a_g$ = *0.35 g*<br>**II zone:** $a_g$ = *0.25 g*<br>**III zone:** $a_g$ = *0.15 g*<br>**IV zone:** $a_g$ = *0.05 g*<br>Performance based design | All Italian territory classified through introduction of a 4th category |
| 14/01/2008 | Uniform hazard spectrum, | Hazard defined on geographical coordinates |

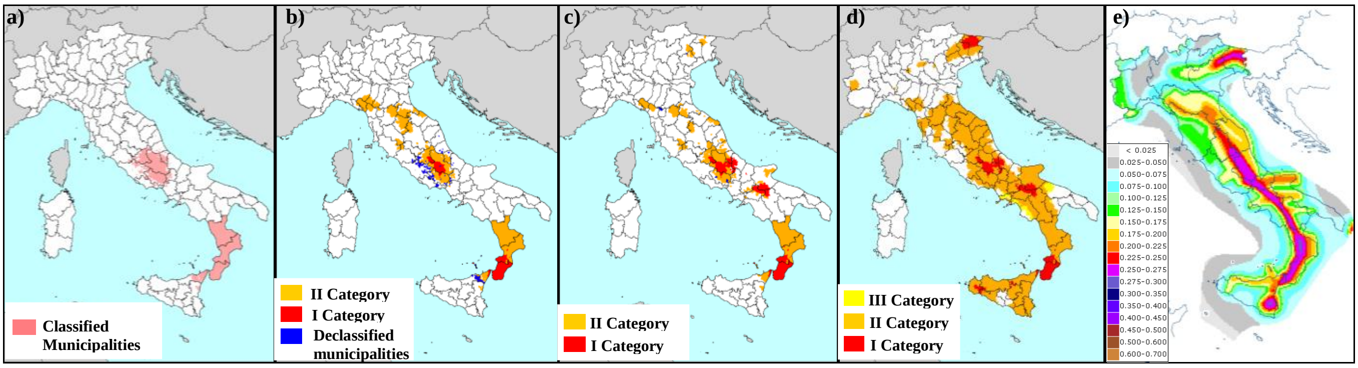


Figure 2. Italian seismicity map after year 1915 (a), 1927 (b), 1962 (c), 1984 (d) and 2009 (e) codes. The latter is based on PGA on rock with 10% exceedance probability in 50 yr .

## 3 NOMINAL DEFICIT AND RISK INDEX

In the following, the proposed approach for seismic risk assessment on large scale is discussed. Following the approach of (Grant *et al.*, 2005 and 2007) and therefore of NZSEE (2003), the prioritization scheme for further, more refined, analyses can be performed assuming that all buildings were designed according to the code enforced at the time of design.

Considering *perfect* code compliance, building capacity can be assumed equal to the seismic demand. In this way, it is possible to define the nominal deficit (NODE) as in Equation 5:

$$NODE = \frac{S_{e,D}(T_{new})}{q} - S_{d,C}(T_{old}) \quad (5)$$

in which $S_{e,D}(T_{new})$ is the elastic seismic demand in terms of spectral acceleration at the fundamental period of the structure, as defined by current seismic code (i.e., NIBC), $q$ is the behaviour factor which allows to transform the elastic acceleration to the design, inelastic, one (see section 3.3), while $S_{a,C}(T_{old})$ is the nominal capacity determined on the basis of the seismic action at the time of design. In the case of the design spectrum (after 1975), it is the spectral acceleration at the fundamental period of the structure; otherwise, it is the seismic coefficient, which is equivalent to assume a spectral shape which is constant with period. Finally, in the case of 2003 code, it is the elastic acceleration divided by the behaviour factor.

It seems reasonable to compare the seismic coefficient and, eventually the inelastic spectral ordinate, with the current demand. In other words, the seismic coefficient and the spectrum defined starting from 1975 are treated as inelastic. The plausibility of this assumption is also remarked by the β coefficient in Equation 1, which can be considered a sort of "inverse" behaviour factor (see Ricci *et al.*, 2011).

NODE can be amplified by a measure of the exposure or loss (L) which, for example, can be expressed in terms of number of occupants, to get a seismic risk index (SRI) as in Equation 6:

$$SRI = L^{\alpha} \cdot NODE = L^{\alpha}\left[\frac{S_{e,D}(T_{new})}{q} - S_{d,C}(T_{old})\right] \qquad (6)$$

where α represents an individual versus social risk index and can assume values ranging between 0 and 1. In fact, if L is, for example, the number of average occupants of the building, it is possible to maximize the social risk posing α equal to 1. Conversely if α is equal to 0 this means to have SRI in terms of individual risk (Grant *et al.*, 2007).

In the proposed approach, as discussed in the following, differently from literature, no common assumptions are made for the building portfolio about the fundamental period, local soil conditions, structural typology and importance factor.

## 3.1 Fundamental period

The proposed index is defined with the aim to measure the nominal deficit between two code-based design demands. In fact, the goal is to quantify the difference between horizontal acceleration according to regulatory codes enforced in different years (today and at the time of construction); this requires the estimation of the fundamental period.

From 1909 to 1975, no response spectrum was defined, but only a single value of seismic coefficient was given, as mentioned.

Since 1975 to 2002 the fundamental period was defined as:

$$T = 0.1 \cdot \frac{H}{\sqrt{B}} \qquad (7)$$

where H is the building height and B is the maximum plan dimension in [m].

Since 2003, T is calculated as follows:

$$T = c_1 \cdot H^{3/4} \qquad (8)$$

where $c_1$ is a coefficient depending on the structural typology; i.e., equal to 0.075 for reinforced concrete (RC), 0.085 for steel, and 0.05 for any other structural type.

Because the spectral accelerations appearing in Equation 5 are proxies for the base shear required at the time of design and current, it may be considered consistent to compare spectral accelerations corresponding to two different fundamental periods computed with old and current formula (Figure 3).

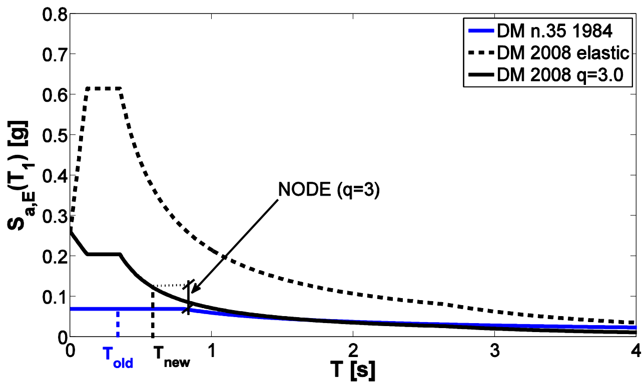


Figure 3. Definition of NODE, for L'Aquila site. For the considered structure, the fundamental period is *0.57 s* according to NIBC and *0.33 s* according to 1984 code.

## 3.2 Importance factor

The importance factor was introduced in 1984 to get a more severe design for those structures with high exposure (e.g., schools, hospitals, or other structures with strategic occupancy).

According to the 1984 code, the importance factor, which multiplies the Equation 1, is equal to 1.4 for most relevant (public) buildings, 1.2 and 1.0 for large occupancy and ordinary structures, respectively. These values remained, substantially, unchanged up to 2008. In fact, NIBC modified the way to account for the occupancy and importance of the building, including the importance factor in the definition of the return period of seismic action. The more important is the structure, the larger is the return period (and, thus, the design seismic intensity).

In the proposed approach the importance factor can be explicitly taken into account for the definition of seismic capacity term $S_{a,C}(T_{old})$ in Equation 5.

### 3.3 Behaviour factor

The definition of the behaviour factor $q$ is the most critical aspect of the method. In fact, the current seismic demand is inelastic, and one of the strongest assumptions of the approach is that demand at the time of design may also be considered inelastic (see Section 2). This is not explicit in the design philosophy of codes pre 1996, meaning that NODE may not be an absolute measure of performance gap but only gives priorities between different structures for which it is applied in a consistent manner.

In any case, in order to compute the current seismic demand it is necessary to define an appropriate behaviour factor. The evaluation $q$ for existing buildings is an important, yet not completely addressed issue. Although, its definition for existing buildings goes beyond the purposes of this paper, it is to mention that NIBC and EC8 provide $q$ values ranging from 1.5 to 3.

It is believed the q-factor should be lower for buildings designed according to older codes. For example, starting from 1996 detailed requirements for local ductility were enforced, so that, for a building designed according to this code, it seems reasonable to assume a larger behavior factor than the one applied to a similar structure designed according an older code. In this way, to a more recent building corresponds a lower value of the nominal deficit.

### 3.4 Local geological conditions

Site classification of subsoil according to NIBC is explicitly considered in the definition of current seismic demand $S_{a,D}(T_{new})$. The inclusion of site effect in the current demand, causes that, in the case of better soil conditions (e.g., type A according to NIBC), the nominal deficit assumes the lower possible value for that site (all other parameters being given).

In analogy with what discussed in section 3.2, the influence of site effect can be explicitly taken into account in the capacity term. For example, starting from 1975 a coefficient $\varepsilon = 1.3$ was considered in the case of “deformable soils”, as reported in Equation 1.

### 3.5 Buildings designed in non-seismic sites

For buildings designed for gravitational loads only; i.e., before the inclusion of construction site in seismic zone, design for lateral resistance was not required and the capacity $S_{a,C}(T_{old})$ reduces to zero. In these cases, NODE is equal to the current seismic demand at the site.

Although it is unreal to suppose that such buildings have no lateral strenght[3], it is intuitively reasonable to assign, comparatively, the largest deficit to the structures designed without any seismic provisions.

In order to take into account for any prescribed lateral resistance, wind design requirements and their evolution with codes (Table 2), are also accounted in the approach.

Being the maximum geometrical dimensions of the building known, it is possible to assess the lateral capacity in terms of wind base shear, as provided by the code enforced at the time of design. If the total mass of the building (dead and live loads, combined according to the seismic code at time of design) is given, it also possible to assess the most demanding action and the corresponding acceleration can be obtained.

The capacity $S_{a,C}(T_{old})$ to be employed in the NODE is, therefore, the maximum between the seismic capacity and the abovementioned wind capacity.

Table 2. Summary of the codes for wind design in Italy considered.

| Year | Code |
|---|---|
| 1967 | CNR-UNI 10012/67 |
| 1978 | DM LL PP 03/10/1978 n.18407 |
| 1982 | DM LL PP 12/02/1982 and Circ. M LL PP n.22631 |
| 1985 | CNR-UNI 10012/85 |
| 1996 | DM LLPP 16/01/1996 and Circ. M LL PP n.156 |
| 2006 | CNR/DT206/06 |
| 2008 | CNR/DT 207/08 |
| 2008 | DM LL PP 14/01/2008 |

### 3.6 Required input data

In the definition of a synthetic index for the prioritization it is important to balance the accuracy of the method with the amount of available information. One of the most important advantages of the approach discussed is its very limited data requirement. In fact, the following information is needed:

1. geographic coordinates of the site and year of the design;
2. height, maximum planar dimension and construction material for the definition of the fundamental period;
3. local site geological (and topographic) conditions;

[3] Some authors (e.g., Bazzurro *et al.* 2005) quantify the seismic coefficient for Italian RC buildings designed for gravitational loads only in 0.08-0.10 g.

4. altitude of the site and other geometrical parameters (for wind capacity assessment).

It is to note that, only data reported at point 1 of the list are strictly necessary. In fact, it is also possible to consider the case in which data related to points 2, 3 or 4 are not available.

If it is not possible to define the fundamental period of the structure, the SRI can be expressed by the PGA deficit (similarly to Grant *et al.*, 2007), rather than that of spectral ordinates, as defined by Equation 9. Figure 4 shows the NODE evaluation in the abovementioned case.

$$NODE = PGA_{new} - PGA_{old} \tag{9}$$

It is considered important, however, to compute homogeneously the nominal deficit, in a way that the same amount of data is considered for all structures, as the strong assumptions of the methods may not enable to provide a meaningful ranking of priorities if NODE is not evaluated consistently within the portfolio of buildings.

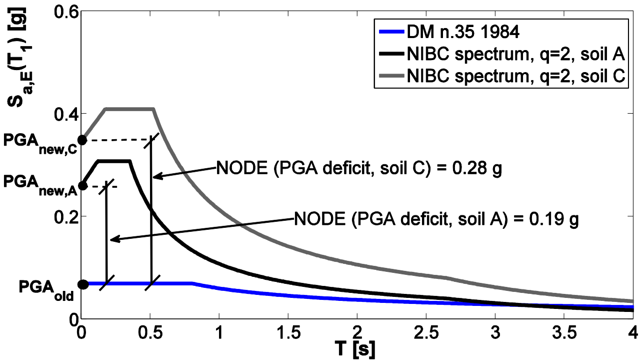


Figure 4. PGA deficit for different site conditions, assuming L'Aquila site and q = 2.

## 4 ASSUMPTIONS AND LIMITATIONS

Nominal deficit, on which the seismic risk index is based on, implies the following assumptions:

1. perfect compliance with regulatory codes enforced at the time of design, which implicitly means applicability to engineering structures only;
2. the current seismic demand is inelastic (see section 3.3) and therefore the demand at the time of design is considered inelastic;
3. except for wind assessment, the methodology assumes zero capacity for those structures in sites not considered as seismically prone at the time of design;
4. live loads are negligible compared to dead loads;
5. NODE can take negative values, in those cases in which the seismic demand at the time of design was greater than the current one; this applies basically in the case of design performed after 2003. In fact, seismic demands were, according to OPCM 3274 (2003), generally larger than those current.

Given these, it is worthwhile to highlight some pros and cons which readily emerge. Regarding the latter:

- NODE compares seismic performance reflecting different design philosophies behind codes at different ages. In fact, most of old Italian codes is based on admissible stress design, which means linear elastic modeling at a material level, without any capacity design principle which underlies current codes; i.e., the second term of the nominal deficit;
- on the other hand, it assumes the capacity at the time being inelastic (i.e., the code horizontal force is assumed to be comparable to linear static design of structures nowadays, which may be incorrect and requires to choose a behavior factor to apply to current seismic elastic demand, which is, to date, a not completely solved issue of research and practice);
- it is a blind prediction based on very poor information, while it is well known that to assess structural seismic, performance of existing structures they have to be known in large detail (e.g., Jalayer *et al.*, 2010; Petruzzelli *et al.*, 2010);
- any overstrength and detailed design rule are neglected, as well as any other systematic deviation from code requirements;
- it does not allow a direct (absolute) estimate of expected loss, yet a comparisons of deficit among a portfolio for which same assumptions can be made;

Regarding the former (pros) apparently there are some, even the strong limitations described:

- it is based on very poor information, which, in the most unfavorable case, may be only the location, year of design, and material/typology, thus applicable at regional scale. Moreover, in many cases, these data are freely available from statistical analysis of building stocks;
- it allows to explicitly account for the evolution of seismic classification of the territory and evolution of codes, which have evolved dramatically in the last century leading to a generally increase in required structural

performances and may be reasonably believed to be the main cause of performance deficit, if any;
- being quantitative, fits with hazard defined at a structural level, and may account explicitly for exposure;
- even if the deficit is biased due to inaccurate assumptions it may be useful to rank priorities if NODE is applied consistently in an homogeneous portfolio.

## 5 NODE V.1.0 (BETA) SOFTWARE

*NODE v.1.0 (beta)* is a software tool developed as a prototype to test the discussed approach to seismic risk prioritization. It was developed in Mathworks MATLAB® environment. As summarized in Figure 5, the software flow follows the steps illustrated above; that is:

1. definition of building location and design year;
2. definition of building dimensions, local geology and behaviour factor;
3. definition of wind design parameter, if any;
4. assessment of NODE;
5. definition of exposure and weight between societal and individual risk;
6. assessment of SRI.

These steps may be deployed by data entry for each specific structures, or loaded automatically by a Microsoft Excel® file for large-portfolio analysis.

### *5.1 Definition location and design year*

To perform the assessment it is necessary to enter geographical coordinates of the site or location name. In both cases the software automatically defines the municipality name, used in Italian seismic classification from 1909 to 2008. Once the design year is entered by the user, the seismic maps for design year code and reference code (2009) are plotted. In this step, the default return period of seismic reference action, equal to 475 years, can be modified according to NIBC.

As discussed in section 3.6, these are the only strictly compulsory data for the assessment; if any other information is not available, it is possible to proceed directly to the assessment (section 5.4). In this case the NODE will be given in terms of PGA deficit.

### *5.2 Definition of building dimensions, local geology, and behaviour factor.*

After step 1, three boxes become editable: the first is relative to the fundamental period; the second to the foundation soil and the last one to the behaviour factor.

The first box contains those parameters necessary for the definition of fundamental period, according to Equation 7 or Equation 8, depending on the design year. Once the “calculate T” button is depressed, the two fundamental periods, at the design year and reference year, are shown.

In the second box local subsoil classification and topographic coefficient, according to NIBC, can be entered.

The last box allows the definition of a user-defined behaviour factor or the use a code-based one, for the current seismic demand. Selecting this latter option, a new window appears containing the NIBC approach for the definition of the behaviour factor for a new building and also other parameters that can be changed for design year. (As discussed in section 3.3, in the NODE evaluation one may employ a behaviour factor defined for existing buildings.)

### *5.3 Definition of wind design parameters*

If the user wants to include in the assessment the evaluation of the horizontal capacity due to wind design, the box “Parameters for wind assessment” must be checked, otherwise it is possible to switch directly to step 4. In the former case, a number of parameters appear, according to the wind code enforced at the time of design. These parameters regard essentially geometric measures of the building, such as height and maximum dimensions (already defined at step 2), pitch number and inclination, aperture percentage and site altitude.

Other parameters, necessary for the evaluation of wind base shear, are automatically calculated by the software according to the code retrieved by design age information.

If the mass of the building is entered, the seismic base shear is also calculated and compared to the wind base shear, in order to assess the most demanding design action among wind and earthquake at the time of construction.

### *5.4 Assessment of SRI*

Once the previous steps are performed, by pressing the “Assessment” button, the SRI index is calculated on the basis of NODE.

In the left box the demand inelastic spectrum and the capacity spectrum (or the seismic coefficient) are plotted. It is now possible to include the exposure and α coefficient, as discussed in previous sections.

Finally, it is possible to export a report file, in txt format, with a summary of all the performed steps and results, and an excel file with response spectrum for the design year and reference year.

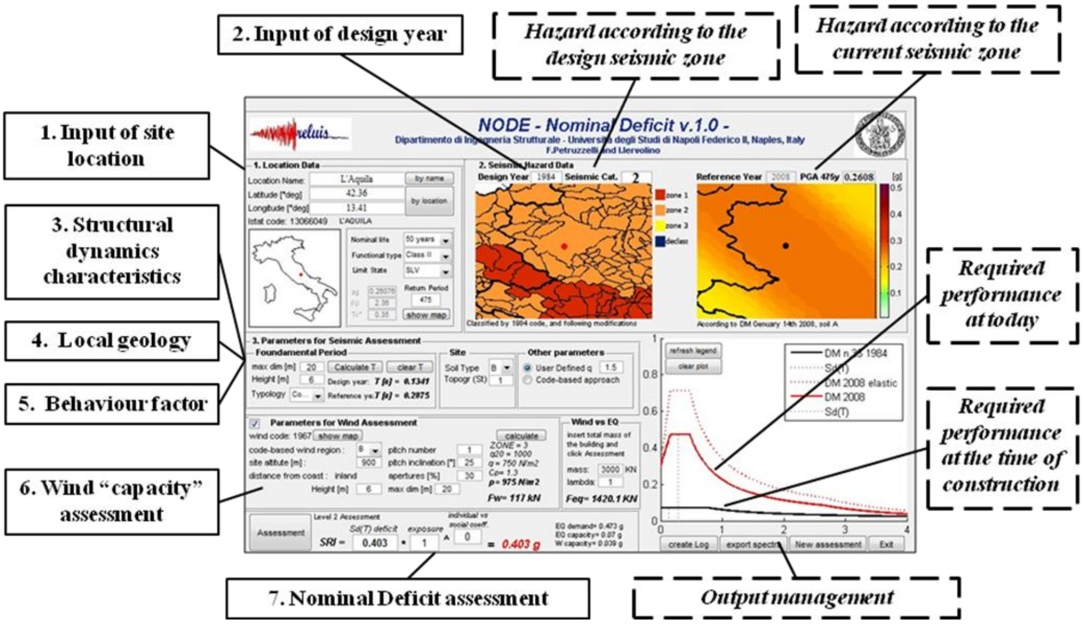


Figure 5. *NODE v.1.0 beta* graphical user interface. In boxes with continuous outline the main steps of the methodology are reported; dashed outline highlight outputs and features.

## 6 CONCLUSIONS

The seismic risk prioritization approach presented in this paper was elaborated on the basis of some previous work in the same direction for a qualitative and relative measure of seismic risk on large (regional up to national) scale, useful for prioritization of interventions on a vast building inventory. It takes into account the current seismic demand and, in a conventional manner, the seismic capacity. In addition, the dynamic characteristics of the structure are taken into account through the definition of fundamental period, as well as the characteristics of local site conditions. Moreover, it may also take into account for the exposure, measurable as number of occupants.

The possibility of including the horizontal wind capacity into the assessment, allows the evaluation of the horizontal capacity for those structures designed in non-seismic zones.

Although, it is based on very strong assumptions and resulting in applicability limitations, it is believed that such an approach could give a rapid measure of relative seismic risk for a portfolio of structures, in a quantitative manner. The balance between accuracy of the assessment and availability of information is achieved by the possibility of including different levels of available information in the assessment.

To test applicability a prototypal software tool implementing, referring to the Italian case, all basic information to address the approach was developed and named NODE v. 1.0 (beta).

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