Assessing the seismic performance of rammed earth walls by using discrete elements

Quoc-Bao Bui*1, Tan-Trung Bui2 and Ali Limam2

Abstract: Rammed earth (RE) is attracting renewed interest throughout the world because of its low embodied energy and its interesting hygric-thermal behavior. Several studies have recently been carried out to investigate this material. However, the seismic behavior of RE walls is still an important subject that needs to be more thoroughly investigated. The present study assesses the seismic performance of RE walls by using the discrete element modeling (DEM) and the nonlinear pushover method. Firstly, nonlinear “force–displacement” curves of the studied wall were obtained by DEM. Secondly, the standard “acceleration–displacement” curves were carried out following Eurocode 8. Thirdly, the above curves were superimposed to determine the intersection point (target point) which enabled to assess the seismic performance of the studied wall in the corresponding conditions (vertical load, seismic zone). The results show that the studied walls can have satisfactory resistance in seismicity zones ranging from “very low” to “moderate” (according to Eurocode 8). For “medium” seismicity zones, the studied structures should only be constructed on A-type soils (very good soil). For B-type soils, wall reinforcement techniques would be necessary. Without special reinforcements, studied RE structures seem unsuitable for “strong” seismicity zones, for all soil types.

Subjects: Geomechanics; Soil Mechanics; Structural Engineering; Waste & Recycling

Keywords: rammed earth; seismic performance; discrete element method; pushover; sustainable development

1. Introduction
Rammed earth (RE) is attracting interest in the context of sustainable development because of its low embodied energy and its interesting hygric-thermal behavior. Several studies have recently been conducted to investigate this material. However, the seismic behavior of RE walls requires more thorough investigation. The first exploratory study on the dynamic characteristics of RE

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PUBLIC INTEREST STATEMENT
Rammed earth (RE) is attracting renewed interest throughout the world because of its “green” characteristics in the context of sustainable development. The present study assesses the seismic performance of RE walls. The results show that the studied walls can have satisfactory resistance in seismicity zones ranging from “very low” to “moderate” (according to Eurocode 8). For “medium” seismicity zones, the RE structures should only be constructed on A-type soils (very good soil). For B-type soils, wall reinforcement techniques would be necessary.
buildings was carried out by Bui, Hans, Morel, and Do (2011). The dynamic characteristics (natural frequencies, mode shapes and the damping) of the studied buildings were identified. These studies also showed that the analytical shear-beam model could reproduce the dynamic behavior of the buildings studied. Then Gomes, Lopes, and de Brito (2011) conducted a numerical study on the seismic resistance of RE constructions in Portugal, but there were still several limitations. Firstly, their model did not analyze the behavior of RE material in detail and there was not a validation step in their numerical model. Secondly, the seismic assessment was conducted using the classical elastic linear equivalent approach, which is less advantageous than the nonlinear approaches. Cheah, Walker, Heath, and Morgan (2012), Hamilton, McBride, and Grill (2006), Miccoli, Müller, and Fontana (2014), Miccoli, Oliveira, Silva, Muller, and Schueremans (2015), and Silva et al. (2013) present various experiments on the shear behavior of several RE walls (stabilized or unstabilized, unreinforced or reinforced). Ciancio and Augarde (2013) studied the out-of-plane of stabilized RE subject to lateral wind force. However, there has not yet been any quantitative study on the seismic performance of RE buildings on the structure scale.

From the viewpoint of earthquake engineering, RE material does not seem favorable. Indeed, the material works essentially in compression and has a very low tensile strength; the walls’ mass is high, which can cause considerable inertial forces during earthquakes. However, according to a number of post-seismic investigations, RE walls present acceptable behaviors. For example, in the Morris and Walker investigation (Morris & Walker, 2011) after the Darfield earthquake (2010, New Zealand, 7.1 on the Richter magnitude scale), only minor cracks were observed in RE walls. This means that when RE buildings are well designed and executed, they can have a satisfactory seismic performance. The present study assesses this performance quantitatively.

It is important to note that the seismic behavior of a RE building depends on several parameters: Earthquake action (seismicity zone, soil type, site factors), the structure’s dynamic characteristics (natural frequencies, modal shapes, damping), and the material’s characteristics (compressive, tensile strengths, Young’s modulus, density). This is why for the same material (RE in this case), the seismic performance of each building may differ depending on its structural characteristics and the quality of its execution. This paper investigates three virtual RE buildings with current designs for RE houses in France and Europe: One story in RE walls, two stories in RE walls, the first story in RE walls and the second story a wooden structure. The RE walls are 50-cm-thick unstabilized RE, built by a pneumatic rammer and their seismic performance is investigated for an almost dry state, several months after their construction (about 2–3% of moisture content, Bui, Morel, Hans, & Meunier, 2009; Bui, Morel, Reddy, & Ghayad, 2009).

The assessment used the nonlinear pushover method, in a numerical model with the discrete element method. One of the main advantages of the numerical approach is the possibility to simulate several pushover tests on a full-scale wall. The wall modeled is in an actual RE house where in situ dynamic measurements were taken during the construction (Bui et al., 2011; Bui, Morel, Hans, et al., 2009). The dynamic characteristics of the wall were measured so that the relevancy of the numerical results could be checked.

2. Pushover method
Traditionally, earthquake-resistant design has been strength-based, using linear elastic analysis. Since inelastic behavior is usually allowed for strong earthquakes, this is not entirely rational. Strength-based design considers inelastic behavior only implicitly. Displacement-based (or deformation-based) design considers inelastic behavior explicitly, using nonlinear analysis (Chopra & Goel, 2002). Displacement-based design recognizes that in an earthquake, inelastic deformation can be greater than that in strength-based. The present paper uses the displacement-based design with the pushover method. It is important to note that the pushover method has not been included in previous French earthquake regulations. It is presented in Eurocode 8 (EN 1998-1:2004, 2004), which has been applied in France and Europe for several years.
The pushover method is a static nonlinear analysis. Firstly, a capacity curve is established (by experiments or numerical models, which enable nonlinear analysis). The seismic force (represented by the shear force at the wall base \( V_b \)) is transformed to spectral acceleration \( S_a \) (Figure 1(a)):

\[
S_a = \frac{V_b}{m}
\]  

(1)

where \( m \) is the mass supported by the wall, and the displacement of the wall top \( u_t \) is replaced by the spectral displacement \( S_D (S_D = u_t) \).

Secondly, the seismic elastic spectrum \( S_a \), which is given in the seismic design code and a function of the structure natural period \( T \), is also transformed in the spectrum \( S_a - S_D \) (Figure 1(b)) by the following relationship:

\[
S_D = S_a / (2\pi / T)^2
\]  

(2)

When the two above curves \( (S_a - S_D) \) are superposed (Figure 1(c)), the intersection point \( (D_1 \) for elastic spectrum or \( D_2 \) for inelastic spectrum) indicates the performance point (or “target point”) that can give information on the damage state of the studied structure. More information on the pushover method can be found in Chopra and Goel (2002) and EN 1998-1:2004 (2004).

As explained above, the pushover method is a nonlinear analysis, so this approach recognizes that inelastic deformation should be taken into account. In general, the design (inelastic) spectrum is obtained by dividing the elastic spectrum by a “behavior factor” \( q \). This is an important parameter in the pushover method; it accounts implicitly for inelastic response, the presence of damping and other force reduce effects, such as period elongation and soil-structure interaction. The behavior factor is defined as the ratio of the elastic acceleration response spectrum expected at a site to that of an inelastic spectrum used for design of a structure (Salvitti & Elnashai, 1996):

\[
q = \frac{S_{a \text{ elastic}}}{S_{a \text{ inelastic}}}
\]  

(3)

The procedure to determine the “rational” \( q \) is quite complex because it depends not only on the material, but also on the structural configuration. This procedure will not be presented in detail here; only typical values proposed in Eurocode 8 will be discussed. \( q \) can be:

- 1 for structures with essentially elastic behavior.
- 1.5 for structures with limited ductility.
- 3 for structures with ductile behavior. Some ductile structures can have \( q = 5 \).
There is not yet a specific value of $q$ for RE structures, but for unreinforced masonry structures, Eurocode 8 authorizes that $q$ is taken to be at least 1.5. If the inelastic spectrum is used, the performance point will be $D_2$ (Figure 1(c)).

3. Discrete element method and parametric studies

3.1. Discrete element method

The explicit discrete element modeling (DEM) based on finite difference principles originated in the early 1970s as the result of landmark work on the progressive movements of rock masses such as 2D rigid block assemblages (Cundall, 1971). This technique was then extended to the modeling of masonry and concrete (Bui, Limam, & Bui, 2014; Lourenço, Oliveira, Roca, & Orduña, 2005). However, to our knowledge, DEM has not yet been used to study RE structures.

The 3DEC code (Itasca, 2011) was used in the present study. The RE wall was modeled as an assemblage of discrete blocks (earthen layers), and the interfaces between earthen layers were modeled by introducing an interface law.

Earthen layers were assumed to be homogeneous and isotropic and were modeled by blocks that were further divided into a finite number of internal elements for stress, strain, and displacement calculations. The failure envelope used in this study was the Mohr–Coulomb criterion with a tension cut-off (Bui, Bui, Limam, & Morel, 2015).

Interfaces between earthen layers were modeled by an interface law between the blocks according to the Mohr–Coulomb interface model with a tension cut-off. This interface constitutive model considers both shear and tensile failure, and interface dilation is included. Further details of the constitutive behaviors of blocks and interfaces can be found in Itasca (2011).

3.2. Parametric studies

In a previous study by the authors (Bui et al., 2015), parametric studies identified 13 parameters necessary for the DEM; the summary of recommendations is presented in Table 1. In that table, $f_c$ is the compressive strength and Young’s modulus was calculated in the elastic part which was from 0 to 20% of the ultimate compressive stress (Bui, Morel, Hans, & Walker, 2014). These recommendations are strengthened with the results presented in other studies that used the finite element method (El Nabouche, Bui, Perrotin, Plé, & Plassiard, 2015; Miccoli, Oliveira, et al., 2015).

| Table 1. Recommended parameters for earthen blocks and interfaces in DEM |
| --- |
| **Earthen blocks** |
| Density $d$ (kg/m³) | Young’s modulus $E_b$ (GPa) | Poisson’s ratio $\nu$ | Tensile strength $f_t$ (MPa) | Cohesion $c$ (kPa) | Friction angle $\phi$ (°) | Dilatancy angle $\Psi$ (°) |
| ~2,000 | (450–500) × $f_c$ | 0.22 | (0.07–0.1) × $f_c$ | (0.07–0.1) × $f_c$ | 45–51° | ~12° |
| Bui, Morel, Hans, et al. (2009) | Bui, Morel, Hans, et al. (2009) | Bui, Morel, et al. (2014) | Bui, Bui, Limam, and Maximilien (2014) | Bui, Bui, et al. (2014), Cheah et al. (2012) | Bui, Bui, et al. (2015) | Bui et al. (2015), Cundall (1971) |
| **Interfaces** |
| Normal stiffness $k_n$ (GPa) | Shear stiffness $k_s$ (GPa) | Tensile strength $f_{t,\text{interface}}$ (MPa) | Cohesion $c_{\text{interface}}$ (kPa) | Friction angle $\phi_{\text{interface}}$ (°) | Dilatancy angle $\Psi$ (°) |
| $\frac{\sqrt{E_b}}{(1 + \nu)}$ | $\frac{\sqrt{E_b}}{(1 + \nu)}$ | 110–150 kPa | 110–150 kPa | 25–45° | ~12° |
| Lourenço et al. (2005) | Lourenço et al. (2005) | Bui et al. (2015) | Bui et al. (2015) | Bui et al. (2015) | Bui et al. (2015) |
According to the proposed values, Bui et al. (2015) obtained useful numerical results by comparing
with experiments in two cases: Loading perpendicular to earthen layers and loading in the diagonal
direction. The DEM application for the lateral loads will be presented in this paper.

4. Seismic capacity of RE walls

4.1. Wall description

The studied wall was constructed in a new RE house in the Rhone-Alpine region, France. It is 50 cm
thick and is an unstabilized RE wall. In situ dynamic measurements were taken on this wall (during
the construction phase, Bui, Morel, Hans, et al., 2009) and on the complete house (after the construc-
tion Bui et al., 2011), so the wall’s dynamic characteristics were determined. A numerical model was
constructed using DEM (Figure 2). The compressive strength determined from the compression tests
on cylindrical samples was used ($f_c = 1.9$ MPa). Young’s modulus, density and Poisson’s ratio were
also measured: 470 MPa (Bui, Morel, Hans, et al., 2009), 20 kN/m$^3$ (Bui & Morel, 2009; Bui, Morel, Hans,
et al., 2009) and 0.22 (Bui, Morel, et al., 2014), respectively.

A parametric study was conducted to identify other characteristics of the earthen blocks and the
interfaces (Bui et al., 2015). The parameters identified are presented in Table 2, which reproduce the
natural frequencies measured on site (Table 3).

4.2. Pushover curve

The wall was simplified but on the safety side: Although the wall has some resistances in the direc-
tion perpendicular to its plane, this resistance is generally ignored for a structural design against

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**Table 2. Parameters of earthen blocks and interfaces used in the DEM model**

|            | Tensile strength $f_t$ | Cohesion $c$ | Friction angle $\phi$ |
|------------|------------------------|--------------|------------------------|
| Earthen blocks | 133 kPa                | 133 kPa      | 45°                    |
| Bui, Bui, et al. (2014) | Bui, Bui, et al. (2014) | Bui, Bui, et al. (2014) |

|            | Normal stiffness $k_n$ | Tensile strength $f_{\text{interface}}$ | Cohesion $c_{\text{interface}}$ | Friction angle $\phi_{\text{interface}}$ |
|------------|------------------------|----------------------------------------|---------------------------------|----------------------------------------|
| Interfaces | 60 GPa/m               | 113 kPa                                | 113 kPa                          | 38°                                    |
| Bui et al. (2015) | 0                      | Bui et al. (2015)                      | Identified                       |
lateral loadings, which is why the L-form wall can be simplified to a line form (Figure 3, left). To obtain the pushover curve, a horizontal force was applied to the wall top and incremented until the wall failed (post-peak).

In building construction, RE walls bear vertical stresses that come from the horizontal elements: the floors or roof (dead loads + live loads). The values of these loads in case of an earthquake will be presented in detail in the next section. Here, to assess the influence of the vertical stresses on the wall’s behavior, three cases of uniform vertical stress on the wall top were simulated: 0, 0.04, and 0.1 MPa. The results are presented in Figure 3, right. It is logical that as the vertical stress increases, the shear performance of the wall increases. However, in a building, when a wall has greater vertical stress due to the horizontal elements (floors, roof), it must also bear a greater mass due to these elements. Therefore, from the earthquake engineering point of view, a wall bearing high vertical loads may be an unfavorable wall.

| Modes | $f_{\text{model}}$ (Hz) | $f_{\text{measured}}$ (Hz) | $f_{\text{model}}/f_{\text{measured}}$ |
|-------|-------------------------|-----------------------------|-------------------------------------|
| 1     | 10.8                    | 10.8                        | 1.00                                |
| 2     | 17.3                    | 18.2                        | 0.95                                |
| 3     | 23.1                    | 24.0                        | 0.96                                |
| 4     | 36.8                    | 36.5                        | 1.01                                |

Table 3. Comparison of frequencies obtained by DEM and by measurements

In building construction, RE walls bear vertical stresses that come from the horizontal elements: the floors or roof (dead loads + live loads). The values of these loads in case of an earthquake will be presented in detail in the next section. Here, to assess the influence of the vertical stresses on the wall’s behavior, three cases of uniform vertical stress on the wall top were simulated: 0, 0.04, and 0.1 MPa. The results are presented in Figure 3, right. It is logical that as the vertical stress increases, the shear performance of the wall increases. However, in a building, when a wall has greater vertical stress due to the horizontal elements (floors, roof), it must also bear a greater mass due to these elements. Therefore, from the earthquake engineering point of view, a wall bearing high vertical loads may be an unfavorable wall.
4.3. Seismic capacity

Because the seismic performance of a RE wall depends on both the vertical stress (favorable role) and the corresponding mass (unfavorable role), several cases should be investigated: Interior or exterior walls; one story, two stories or more; and the value of the loads from the horizontal elements (floor, roof), which depend on the dimensions of these elements between the bearing walls (influenced zones). For a general assessment of the seismic performance of the RE buildings in France (and Europe), three virtual RE buildings are considered:

- Only one story and all bearing walls are in RE (this is the case of the measured house).
- Two stories in RE walls and all bearing walls are in RE.
- First story with all bearing walls in RE and second story in a wooden structure.

The floor (or roof) space between bearing walls measures 6 m × 6 m, but the wall length is always 2.5 m, like the real house studied. The vertical elements between the walls and on the facades are light elements (wooden and glass infill). In general, the bearing RE walls have lengths greater than or equal to 2.5 m, so the length chosen also tends toward a greater safety. The wall height is 2.7 m, the same as the house studied.

In a seismic design, in addition to the earthquake load, other loads should also be taken into account; they are given according to Eurocode 8:

\[
\sum \text{Dead load} + \varphi \cdot \Psi_{2i} \cdot \sum \text{Live load} = \text{Target displacement} \div \text{storey height}
\]

(4)

where \( \Psi_{2i} = 0.3; \varphi = 1 \) for the roof and 0.5 for the floors (for residential, office, and commercial buildings).

In this paper, only residential buildings are studied; therefore, according to Eurocode 1, the live loads composed of an exploitation load of 1.5 kN/m² and the light partitions of 0.5 kN/m². The usual self weight of the wooden floor was \( g = 1.1 \) kN/m². The synthesis of the mass and vertical stress due to the loads on the floors and roofs is given in Table 4.

Pushover simulations were performed in the DEM model for each wall to obtain the capacity curves. Then the demand spectrum was constructed for important class II (current buildings) and the soil type A under the building (Figure 5). Soil type A corresponds to rock or a very good soil (mean velocity of the shear waves \( v_{S2} > 800 \) m/s). Indeed, RE buildings in France are usually constructed on soil A or B. Soil type B will be discussed below.

From the capacity curves and the demand spectra, the intersection target points can be determined for each case (wall type and seismicity zone). Then the inter-story drift can also be determined:

Inter-storey drift = target displacement/storey height

(5)

| Building type          | Wall          | Added vertical stress (MPa) | Added mass (ton) |
|------------------------|---------------|-----------------------------|------------------|
| One story in RE        | Exterior wall | 0.025                       | 9.8              |
|                        | Interior wall | 0.05                        | 12.9             |
| Two stories in RE      | Exterior wall | 0.04                        | 18.5             |
|                        | Interior wall | 0.08                        | 23.6             |
| First RE + second wood | Exterior wall | 0.04                        | 12.0             |
|                        | Interior wall | 0.08                        | 17.0             |
The inter-story drift values determined are presented in Table 5.

To assess the seismic performance of the walls studied, a criterion must be chosen. For inter-story drift, Calvi (1999) proposed three damage limit states (LS) for masonry structures (Figure 6):

- **LS2**—Minor structural damage and/or moderate nonstructural damage; the building can be utilized after the earthquake, without any need for significant strengthening and repair to structural elements. The suggested drift limit is 0.1%.
- **LS3**—Significant structural damage and extensive nonstructural damage. The building cannot be used after the earthquake without significant repair. Still, repair and strengthening are feasible. The suggested drift limit is 0.3%.
- **LS4**—Collapse; repairing the building is neither possible nor economically reasonable. The structure will have to be demolished after the earthquake. Beyond this, LS global collapse with danger for human life has to be expected. The suggested drift limit is 0.5%.

### Table 5. Damage assessment (with Inter-story drift, %) for soil A

| Seismicity zone | Wall alone | One story RE | First RE + Second wood | Two stories RE |
|----------------|------------|--------------|------------------------|----------------|
|                | Ext. wall  | Int. wall    | Ext. wall              | Int. wall      | Ext. wall  | Int. wall    |
| Very low       | Slight (0.009) | Slight (0.012) | Slight (0.013) | Slight (0.012) | Slight (0.013) | Slight (0.012) |
| Low            | Slight (0.011) | Slight (0.022) | Slight (0.023) | Slight (0.024) | Slight (0.025) | Slight (0.026) |
| Moderate       | Slight (0.018) | Slight (0.044) | Slight (0.038) | Slight (0.088) | Moderate (0.111) | Moderate (0.125) |
| Medium         | Slight (0.062) | Moderate (0.101) | Moderate (0.108) | Moderate (0.185) | Moderate (0.202) | Moderate (0.227) |
| Strong         | Complete (∞)  | Complete (∞)  | Complete (∞) | Complete (∞) | Complete (∞) | Complete (∞) |
Following the above description, it is suggested that depending on the seismic demand, if a building has a behavior that does not exceed LS3, it can be considered satisfactory. Using the above criteria, the damage states of the wall studied for each corresponding seismicity zone can be determined and are presented in Table 5. It can be observed that except for the strong seismicity zones, RE buildings on an A-type soil can present satisfactory seismic performance.

Following the well-known equation (2) in the dynamic of structures ($S_a = \omega^2 S_d$), the initial slope of the capacity curve corresponds to $\omega^2$ (with $\omega = 2\pi f$). That is why from this curve, the frequency $f$ can be determined; the result is presented in Figure 7, noted “$f_{\text{numerical}}$”. On this figure, $h$ is the height of the structure. From this figure, it is interesting to note that the obtained inelastic frequencies have the same order of magnitude as the frequency obtained using other approaches: Empirical formula (Eurocode 8), noted “$f_{\text{EC8}}$”; analytical with shear beam theory (Bui et al., 2011), noted “$f_{\text{shear beam}}$”; and in situ measurement (Bui et al., 2011), noted “$f_{\text{experimental}}$”. This confirms the relevancy of the models used. It is logical that the inelastic frequencies have values slightly lower than the elastic frequency values, except the case of first story in RE and the second story in wooden structure. The reason may be that this mixed structure does not follow the relationships established for the current structures (one material for all walls).

In the same procedure, RE buildings on a B-type soil were also assessed. The B-type soil following Eurocode 8 corresponds to good soil (shear wave velocity $v_{s,30} = 360–800$ m/s). The results are presented in Table 6.

It can be observed that for soil B, without special reinforcement, RE buildings in the investigated configuration do not seem adapted for medium and strong seismicity zones.
4.4. Discussion

For the configurations studied in this paper, the results show that for “very low” to “moderate” seismicity zones: Unreinforced RE buildings can be constructed on A and B soils. For “medium” seismicity zones, only soil A is acceptable. For other seismicity zones with other soil types, RE constructions are not authorized without appropriate reinforcements. This result is similar to a condition imposed in Eurocode 8 for unreinforced masonry:

\[ a_g \cdot S < 2 \text{m/s}^2 \]  

where \( a_g \) is the design horizontal acceleration and \( S \) depends on soil types (Appendix A).

Indeed, from Equation (6), for “very low” to “moderate” seismicity zones, unreinforced masonry buildings can be constructed on soils A and B. For “medium” seismicity zones, only soil A is appropriate. For other seismicity zones with other soil types, unreinforced masonry is not authorized. This condition is similar to the results above obtained in this study. Therefore, this condition in Eurocode 8 for unreinforced masonry seems applicable also for RE buildings studied in this paper. However, in practice, if the RE wall length increases (more than 2.5 m for a 6 m span like the house studied in this study), the wall’s seismic performance may also increase.

5. Conclusions and prospects

From a general point of view, the static nonlinear method gives more pertinent information than the static elastic method. For structures in which the dynamic behavior is dominated by the first vibrational mode (e.g. low-rise buildings), the pushover technique gives good estimations of displacements and can give useful information to assess the building’s seismic vulnerability.

The results confirm that the mass of the upper stories does not play a favorable role because on the one hand it increases the vertical stress on the walls but on the other hand, it increases also the seismic action (inertia effect).

For the configurations studied in this paper, the results show that for “very low” to “moderate” seismicity zones: Unreinforced RE buildings can be constructed on A and B soils. For “medium” seismicity zones, only soil A is acceptable. For other seismicity zones with other soil types, RE constructions are not authorized without appropriate reinforcements. This result shows that the condition imposed in Eurocode 8 for unreinforced masonry seems applicable also for RE buildings.

This paper concentrated on the in-plane behavior of RE walls. Their out-of-plane behavior will also be studied in future studies. Other reinforcement techniques will also be tried to check their relevancy for RE buildings.
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Appendix A

For unreinforced masonry, Eurocode 8 imposes the following condition: \( a_g \cdot S < 2 \, \text{m/s}^2 \)

where

\[ a_g : \text{Design horizontal acceleration:} \quad a_g = g_I \cdot a_{gr} \]

where \( a_{gr} : \text{Ground acceleration referenced according to the seismicity zones} \)

| Seismicity zones | \( a_{gr} \) (m/s^2) |
|------------------|---------------------|
| 1 (very low)     | 0.4                 |
| 2 (low)          | 0.7                 |
| 3 (moderate)     | 1.1                 |
| 4 (medium)       | 1.6                 |
| 5 (strong)       | 3                   |

\( g_I : \text{Important factor. For category II (current buildings, } h < 28 \, \text{m}): g_I = 1; \text{ category III (schools, meeting halls, etc.): } g_I = 1.2 \)

\( S : \text{Depends on soil types} \)

| Soil types                                      | \( S \) (zones 1–4) |
|------------------------------------------------|---------------------|
| A (rock)                                        | 1                   |
| B (stiff deposit of sand, gravel or over consolidated clay) | 1.35                |
| C (deposit of medium-density sand)              | 1.5                 |
| D (deposit of noncohesive soils from low to medium density) | 1.6                 |
| E (alluvium)                                    | 1.8                 |