Seismic resistance of a reinforced concrete building before and after retrofitting
Part I: The existing building

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Abstract

In this work the seismic resistance of a reinforced concrete building, before and after a seismic retrofitting, is evaluated. The building was built in the late seventies in Eastern Sicily, a high seismic area where regulations have been enforced only since 1981, therefore it was not designed to resist earthquake loading. Recently a retrofitting project for the considered building has been developed by a professional engineers team. The project includes both a partial reinforcement of the existing structure and a base isolation. In this part I the problem of the evaluation of the seismic resistance of the existing building is addressed by means of three-dimensional nonlinear analyses as well as of simplified procedures. The building has been modelled as a space-frame in which the nonlinear behaviour has been introduced by means of lumped plasticity elements. The performed three-dimensional nonlinear analyses have also been used for the definition of equivalent nonlinear single-degree-of-freedom systems. Such simplified systems allow the estimation the level of peak ground acceleration (PGA) which the building can withstand according to a prescribed design spectral shape and therefore the evaluation of its seismic resistance with reference to specified seismicity classes and local site conditions. In the part II an advancement of the above-mentioned procedure, specialised for buildings retrofitted by means of traditional and innovative techniques, is described and applied to the case-study building.

Keywords: seismic engineering, reinforced concrete buildings, seismic resistance, seismic retrofitting, base isolation, shear walls, nonlinear dynamics.
1 Introduction

The safeguard of structures against the damaging effects of earthquakes can be achieved either strengthening the structure or using an alternative earthquake-resistant design strategy. Over the last twenty years the developments in computational structural dynamics coupled with technological improvement and with the experience of the more recent earthquakes caused significant advances in the knowledge of earthquake engineering problems and design. The effects of earthquakes on existing seismic resistant structures had shown that the design philosophy based on ductility must be critically reviewed because the recourse of ductility causes significant structural damage and, as a consequence, very high economic losses [1]. Therefore an innovative approach based on passive energy dissipation systems and base-isolation systems is certainly preferable to the classical approach based on structural strengthening and ductility. These novel techniques are nowadays widely used in seismic areas and several examples of passive controlled structures have already been constructed all over the world. In many cases this innovative approach has been successfully applied for the seismic retrofitting of existing structures for which the study of seismic vulnerability and seismic resistance is preliminary to the subsequent seismic rehabilitation.

This work deals with the evaluation of the seismic resistance of a reinforced concrete building that originally was not designed to resist earthquake loading, before and after a seismic retrofitting. The building has been built in the late seventies in Eastern Sicily, a high seismic area where regulations have been enforced only in 1981. Recently a retrofitting project for the considered building has been developed by a professional engineers team [2]. The project includes both a partial reinforcement of the existing structure and the base-isolation of the building.

In this part I the problem of the evaluation of the seismic resistance of the existing building is addressed according to simplified procedures presented by Oliveto et al. in previous works [3, 4]. The building has been modelled as a three-dimensional frame in which the nonlinear behaviour of the structure has been introduced by means of lumped plasticity elements. Nonlinear static analyses have been carried out in order to attain to the definition of equivalent single-degree-of-freedom systems. Such simplified systems allow the estimation of the level of peak ground acceleration (PGA) which the building can withstand according to a prescribed design spectral shape and therefore the evaluation of its seismic resistance with reference to specified seismicity classes and local site conditions.

2 Description of the building

The building under investigation is representative of many reinforced concrete buildings constructed in Eastern Sicily in the 1960s and the 1970s and designed for vertical loads only. The building has four storeys and a smaller top storey, with the ground level up-elevated by means of squat columns. Figure 1a shows a
recent picture of the front view of the building, while in figure 1b a particular of the foundation and of the squat columns is reported. The typical rectangular floor plan, shown in figure 2, is 24.70 metres long and 8.50 metres wide.

As it can be seen in figure 2, a geometrical symmetry axis can be identified in the transverse direction. Nevertheless, due to the presence of knee-type beams in the stair-cases, the structure is not perfectly symmetric. The inter-storey height is equal to 3.30 metres for the four main storeys, while the top storey is positioned 2.30 metres above the fourth floor slab. The total height of the structure is therefore equal to 15.50 metres.

The structural skeleton may be assimilated to a spatial framework with special constraints provided by the floor slabs. In the longitudinal direction it is possible to identify three main plane frameworks while the transverse direction it is possible to identify only two complete plane-frames at the two ends of the building. This is typical of reinforced concrete buildings not designed for earthquake action. Therefore the structural layout suggests the use of a space-frame model and the inclusion of the physical constraints provided by the floor slabs.

Figure 1: The considered building: (a) a recent picture of the front view of the building; (b) particular of the foundation and of the squat columns.

Figure 2: Typical floor plan with column numbers.
To the aim to obtain an insight into the real conditions of the building, some experimental investigations have been conducted. These have been included destructive compression tests on concrete samples and traction tests on steel rebars. From the results of the experimental tests, the characteristic values of the mechanical properties of the materials reported in table 1 have been found. It is worth to notice that the measured concrete characteristic cubic strength results about 48% lower than that indicated in the original design.

2.1 Dynamical characteristics of the building

The mass properties of the building have been evaluated, by means of the lumped mass method, including the gravity loads and one third of the characteristic value of the live loads. The floor masses and the total mass of the building are reported in table 2.

The first three mode shapes and the corresponding periods are reported in figure 3. It is apparent that the first mode corresponds to a translational motion in the longitudinal direction while the second mode corresponds to a torsional motion of the building. The third mode is mainly a translational motion in the transversal direction.

3 The inelastic space-frame model

For the evaluation of the global nonlinear behaviour of the building, a simplified three-dimensional lumped plasticity model have been considered. In the model the plastic deformation can occur only in pre-defined cross-sections (critical sections) located where the formation of plastic hinges is expected. The only plastic interaction that has been considered is the one between the static axial force and the dynamical bending moments. The evaluated bending moment/curvature relationship of each critical section has been replaced by a bi-linear elastic-perfectly plastic one with yield and ultimate moments coincident. This idealised behaviour allows for an easy definition of the maximum permissible ductility or available ductility of the cross-section.

<table>
<thead>
<tr>
<th>Concrete 28 day characteristic cubic strength, $f_{cu,28}$ (N/mm²)</th>
<th>13</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete ultimate design strength, 0.83 $f_{cu,28}$ (N/mm²)</td>
<td>10.79</td>
</tr>
<tr>
<td>Elastic modulus for concrete, $E_{c,28}$ (kN/mm²)</td>
<td>20.55</td>
</tr>
<tr>
<td>Concrete maximum parabolic strain, $\varepsilon_{co}$</td>
<td>0.2%</td>
</tr>
<tr>
<td>Concrete ultimate strain, $\varepsilon_{cu}$</td>
<td>0.35%</td>
</tr>
<tr>
<td>Steel type</td>
<td>Fe B 38 k</td>
</tr>
<tr>
<td>Steel characteristic ultimate strength, $f_{su}$ (N/mm²)</td>
<td>450</td>
</tr>
<tr>
<td>Steel characteristic yield strength, $f_{sy}$ (N/mm²)</td>
<td>375</td>
</tr>
<tr>
<td>Elastic modulus for steel, $E_{s}$ (kN/mm²)</td>
<td>206</td>
</tr>
<tr>
<td>Steel ultimate strain, $\varepsilon_{su}$</td>
<td>1%</td>
</tr>
</tbody>
</table>

Table 1: Mechanical properties of concrete and steel.
Table 2: Total mass per floor.

<table>
<thead>
<tr>
<th>Floor</th>
<th>Ground</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>Whole building</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass (kN(s^2/m))</td>
<td>166</td>
<td>157</td>
<td>154</td>
<td>154</td>
<td>112</td>
<td>10</td>
<td>587</td>
</tr>
</tbody>
</table>

3.1 Pushover analyses

It is well known that the most rigorous procedure for the evaluation of the seismic response of a structure is the nonlinear response history analysis \( (RHA) \) to a specified ground motion; however in the current engineering practice the nonlinear static procedure \( (NSP) \), or pushover analysis, is often preferred \([5, 6]\). According to this procedure the seismic demands are computed by nonlinear static analysis of the structure subjected to monotonically increasing lateral force with invariant height-wise distribution. More advanced pushover analysis procedures that try to take into account the higher modes contribution to the response can be found in the literature \([7, 8, 9, 10, 11, 12]\), however for low- and medium-rise structures it has been proved that pushover analyses based on an invariant distribution associated to the fundamental mode of vibration lead to satisfactory predictions. Therefore, for the purposes of the present study, for each of the main directions of the building, an increasing force distribution proportional to the corresponding fundamental mode has been considered. In the following the results of the performed pushover analyses will be reported in terms of significant parameters that are widely used in the earthquake engineering practice: namely the seismic coefficient, that is given by the ratio between the base shear and the total weight of the building, and the inter-storey drifts.

Two pushover analyses have been performed, one in the transverse direction and one in the longitudinal direction. The force distributions of the corresponding fundamental modes have been considered. It is worth noticing that the modal participating masses associated to those modes are respectively the 82.5% and the 76.5% of the total mass of the building. This is in line with the FEMA guidelines \([5, 6]\) and ensures that the higher mode contributions are not relevant. The lateral force distributions have been applied at the joints of the physical model of the framework. The resultants of these forces at each floor have been calculated and reported in table 3, for both the longitudinal and the transverse directions, where they are compared with the lateral forces usually provided by seismic codes.

\[ T_1 = 0.94 \text{ s} \quad T_2 = 0.87 \text{ s} \quad T_3 = 0.71 \text{ s} \]

Figure 3: Spatial views of the first three modes of vibration.
Table 3: Resultants of the force distributions for each floor.

<table>
<thead>
<tr>
<th>Floor</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse direction, $V_i/V_b$</td>
<td>0.072</td>
<td>0.198</td>
<td>0.320</td>
<td>0.380</td>
<td>0.029</td>
</tr>
<tr>
<td>Longitudinal direction, $V_i/V_b$</td>
<td>0.096</td>
<td>0.218</td>
<td>0.309</td>
<td>0.332</td>
<td>0.045</td>
</tr>
<tr>
<td>Seismic codes, $V_i/V_b$</td>
<td>0.102</td>
<td>0.201</td>
<td>0.291</td>
<td>0.373</td>
<td>0.032</td>
</tr>
</tbody>
</table>

Figure 4: Relative storey-drift as a function of the base-shear coefficient: (a) transverse direction; (b) longitudinal direction.

The results of the pushover analyses in terms of the base-shear coefficient against the storey-drifts are reported in figure 4a for the transverse direction and in figure 4b for the longitudinal direction.

The pushover analyses have been conducted to far past the point where the first cross-section reaches its ultimate state and therefore breaks down. This point has been evaluated a posteriori and the pushover curves beyond this point represent only an ideal, unrealistic, elastic-indefinitely plastic behaviour. In the figures this idealised behaviour is distinguished from the realistic behaviour up to the point when the first cross-section rupture occurs. This condition has been taken as characteristic of the conventional collapse of the building.

As has been pointed out before, squat columns are present between the ground floor slab and the foundation. These columns, with cross-section dimensions much larger than the others, have been introduced in the model. Nevertheless they exhibit an elastic behaviour up to the conventional collapse of the building, therefore the base-shear coefficient reported in the following figures has been calculated as the ratio between the resultant shear-force over the ground floor slab and the total weight of the portion of the building above the ground level. The behaviour of the structure up to collapse may be distinguished in three different phases. The first one, of ‘elastic behaviour’, identifies the structural response up to the formation of the first plastic hinge. The second one, of ‘moderate plastic behaviour’, is characterised by moderate inelastic deformations and quasi-linear behaviour. The third one, of ‘strong plastic...
behaviour’, is characterised by the formation of a large number of plastic hinges and large inelastic deformations.

When the excitation acts along the transverse direction, as shown in figure 4a, the first plastic hinge is activated for $C_b=0.036$. For lower values of $C_b$ the structure exhibits therefore elastic behaviour. As the base-shear coefficient increases a larger number of plastic hinges is activated. This plastic behaviour can be considered moderate for $C_b<0.06$, while it becomes relevant for $C_b>0.06$. In the latter case a significant damage of the structural elements can be observed. By comparing the allowable ductility of each critical cross-section with the requested plastic deformation it can be pointed out that the first plastic hinge that reaches the collapse condition is located at the end of a beam of the second level when the base-shear coefficient equals 0.088 and the maximum storey-drift equals the 0.32% of the inter-storey height.

When the load is applied along the longitudinal direction, as can be seen in figure 4b, the formation of the first plastic hinge corresponds to $C_b=0.041$. For larger values of the base-shear coefficient some structural members will experience plastic deformation. Nevertheless, for $C_b<0.10$ the plastic deformation is rather small. Only for $C_b>0.10$ a relevant plastic deformation, and therefore a significant damage, should be considered up to the conventional collapse condition that is verified for $C_b=0.160$ and for the maximum storey-drift equal to the 0.68% of the inter-storey height. It can be pointed out that the first plastic hinge that reaches the collapse condition is located at the end of a beam of the second level.

Table 4: Number of plastic hinges and storey-drifts for three values of the base-shear coefficient for each loading direction.

<table>
<thead>
<tr>
<th>Level</th>
<th>Structural members</th>
<th>Transverse direction</th>
<th>Longitudinal direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$C_b = 0.036$</td>
<td>$C_b = 0.06$</td>
<td>$C_b = 0.088$</td>
</tr>
<tr>
<td></td>
<td>Storey-drift $\Delta u/h_i$ (%)</td>
<td>Plastic hinges</td>
<td>Storey-drift $\Delta u/h_i$ (%)</td>
</tr>
<tr>
<td>1</td>
<td>Beams</td>
<td>0.028</td>
<td>0.060</td>
</tr>
<tr>
<td></td>
<td>Columns</td>
<td>0.053</td>
<td>0.131</td>
</tr>
<tr>
<td></td>
<td>All</td>
<td>0.050</td>
<td>0.106</td>
</tr>
<tr>
<td>2</td>
<td>Beams</td>
<td>0.061</td>
<td>0.135</td>
</tr>
<tr>
<td></td>
<td>Columns</td>
<td>0.050</td>
<td>0.106</td>
</tr>
<tr>
<td></td>
<td>All</td>
<td>0.061</td>
<td>0.135</td>
</tr>
<tr>
<td>3</td>
<td>Beams</td>
<td>0.050</td>
<td>0.106</td>
</tr>
<tr>
<td></td>
<td>Columns</td>
<td>0.050</td>
<td>0.106</td>
</tr>
<tr>
<td></td>
<td>All</td>
<td>0.061</td>
<td>0.135</td>
</tr>
</tbody>
</table>

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In table 4 the values of storey-drifts and the numbers of plastic hinges, in the beams and in the columns, corresponding to the three values of the base shear coefficient are reported for each floor. From the analysis of the results it may be observed that when the excitation acts in the transverse direction most of the plastic hinges occur in the columns. On the contrary when the excitation acts in the longitudinal direction very few plastic hinges occur in the columns, while almost the total number of active plastic hinges occur in the beams.

In the collapse conditions the total number of plastic hinges is equal to 104 when the excitation acts in the transverse direction and to 131 when the excitation acts in the longitudinal direction. In the first case the number of plastic hinges at collapse is 37 in the beams and 67 in the columns, while in the second case 115 plastic hinges occur in the beams and only 16 in the columns. Furthermore in the transverse direction the number of plastic hinges decreases from the first storey to the top storey, while in the longitudinal direction the distribution of plastic hinges is almost constant in the first three storeys and only 3 plastic hinges occur in the fourth storey.

Finally it is interesting from a practical point of view to consider the storey-drifts at collapse. When the excitation acts in the transverse direction, the storey-drifts at collapse vary from a minimum of 0.236% of the storey height for the first storey to a maximum of 0.330% for the top storey. For excitation in the longitudinal direction, the minimum storey-drift is equal to 0.219%, while the maximum is equal to 0.682%.

The above commented results clearly show that the building is weaker in the transverse direction, being characterised by a collapse base-shear coefficient 55% lower than the longitudinal direction. Furthermore the values of the inter-storey drifts corresponding to the collapse condition had shown that in the longitudinal direction, in which most of the plastic hinges occur in the beams, the building is more ductile than in the transverse direction. This conclusion will be better summarised in the following by means of the definition of equivalent single-degree-of-freedom systems.

4 Equivalent single-degree-of-freedom systems

The three-dimensional lumped plasticity model of the real building is sufficiently simple for carrying pushover and nonlinear dynamic analyses with a reduced computational effort. Nevertheless, in order to establish the seismic resistance capacity of the building in terms of commonly used ground motion parameters, it is convenient to replace the actual model with an equivalent single-degree-of-freedom system.

As reported in previous works [3, 4], various procedures that allow the definition of a single-degree-of-freedom system equivalent to the building can be found in the literature [5, 6]. Here a procedure based on the energy equivalence [3] is applied. As the base-shear is a fundamental parameter used in seismic engineering and in seismic codes, the base-shear of the actual model is used as the force parameter for the single-degree-of-freedom equivalent system. The corresponding displacement is calculated on the principle of equality of works.
for the actual model and the equivalent single-degree-of-freedom system.

From each of the two single-degree-of-freedom nonlinear curves, one for each of the two principal directions of the building, a bi-linear relationship can be obtained and therefore an effective period for the structure in the given direction can be defined:

$$T_{eff} = 2\pi \sqrt{\frac{M}{k_{eff}}}$$  \hspace{1cm} (1)

where $M$ is the mass of the single-degree-of-freedom model that is equal to the total mass of the building and $k_{eff}$ if the stiffness of the first branch of the bi-linear relationship (effective stiffness). It can be noticed that the bi-linear system is used only for the definition of $k_{eff}$, while in the subsequent dynamic analyses the nonlinear equivalent system will be used.

Furthermore the equivalent single-degree-of-freedom system allows the definition of a global ductility capacity parameter [4]:

$$\mu = \frac{1}{2} \left( 1 + \frac{1}{h} \right) \cdot \frac{u_c \cdot k_{eff}}{F_c} + \frac{1}{2} \left( 1 - \frac{1}{h} \right)$$  \hspace{1cm} (2)

where $u_c$ is the collapse displacement, $F_c$ is the collapse force and $h = F_c/F_y = C_{b,c}/C_{b,y}$ is the force hardening ratio.

The results of the application of the described procedure for the definition of the single-degree-of-freedom equivalent systems are reported in figures 5. Specifically, figure 5a shows the pushover curves for the four storeys of the building, the force/displacement curve for the equivalent system and the corresponding bi-linear relationship when the excitation acts along the transverse direction. Figure 5b reports the same results for the longitudinal excitation.

![Figure 5: Definition of the single-degree-of-freedom equivalent system: transverse (a) and longitudinal (b) direction.](image)
Table 6: Main results of the definition of the equivalent system.

<table>
<thead>
<tr>
<th>Direction</th>
<th>Transverse</th>
<th>Longitudinal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yielding $C_b$ for the bi-linear system, $C_{b,y}$</td>
<td>0.079</td>
<td>0.124</td>
</tr>
<tr>
<td>Effective yielding base-shear coefficient, $C_{b,y'}$</td>
<td>0.071</td>
<td>0.110</td>
</tr>
<tr>
<td>Collapse base-shear coefficient, $C_{b,c}$</td>
<td>0.088</td>
<td>0.160</td>
</tr>
<tr>
<td>Effective yielding displacement, $u_y$ (cm)</td>
<td>1.43</td>
<td>2.15</td>
</tr>
<tr>
<td>Collapse displacement, $u_c$ (cm)</td>
<td>3.16</td>
<td>4.78</td>
</tr>
<tr>
<td>Effective period, $T_{eff}$ (s)</td>
<td>0.85</td>
<td>0.84</td>
</tr>
<tr>
<td>Hardening force ratio, $h = C_{b,c}/C_{b,y}$</td>
<td>1.12</td>
<td>1.30</td>
</tr>
<tr>
<td>Global ductility, $\mu$</td>
<td>1.98</td>
<td>1.92</td>
</tr>
</tbody>
</table>

The main numerical parameters of the application of the procedure are summarised in table 6. Among these, the most relevant are the effective period $T_{eff}$ and the global ductility parameter $\mu$.

5 Seismic resistance

The results obtained by means of the pushover analyses, together with the local site conditions, allows us to estimate the seismic resistance with reference to the expected earthquake at the construction site.

The seismic resistance will be expressed in terms of the maximum peak ground acceleration that the building can withstand according to a prescribed response spectrum (collapse-PGA). The estimation of the collapse-PGA will be achieved in two different ways, described in the following paragraphs, characterised by different computational cost. The first method is very simple and is based on the use of inelastic design spectra, the second procedure is based on the results of many nonlinear dynamic analyses conducted on the nonlinear equivalent systems subjected to artificially generated ground motions compatible with the prescribed pseudo-acceleration design spectrum.

5.1 Estimation of the collapse-PGA by means of inelastic design spectra

The site seismicity is usually specified by means of design spectra in terms of pseudo-acceleration. For example, the inelastic design spectrum assigned by Eurocode 8 [13] provides the following values of the pseudo-acceleration:

$$S_a = a_g S \beta_0 f(T_{eff}, S) / q$$  \hspace{1cm} (3)

where

- $a_g$ is the peak ground acceleration expected at the construction site;
- $S$ is a local site condition factor;
- $\beta_0$ is a constant amplification factor equal to 2.5;
- $q$ is the behaviour factor that depends mainly on the ductility resources of the structure;
is the spectral shape function that depends on the fundamental period of the structure, on the behaviour factor and on the construction site characteristics.

The pseudo-acceleration $S_g$ is related to the collapse base-shear coefficient $C_{b,c}$ obtained from the pushover analyses

$$C_{b,c} = \frac{S_g}{g}$$

thus equation (3) can be rewritten with reference to the collapse condition:

$$C_{b,c} = \frac{a_{g,b} S \beta_0 f(T_{eff}, S)}{g}$$

where the only unknown is the minimum peak ground acceleration $a_{g,b}$ that takes the building to the collapse condition (collapse-PGA). By solving equation (5) for $a_{g,b}$ it yields

$$a_{g,b} = \frac{S \beta_0 f(T_{eff}, S)}{q C_{b,c}}.$$  

For the behaviour factor $q$ the following expressions, adapted from Newmark & Hall [14, 15], can be considered [16]:

$$q(T, \mu) = \begin{cases} 
\sqrt{2\mu - 1} & \text{for} \quad T_B < T \leq T_C \\
\mu & \text{for} \quad T > T_C 
\end{cases}$$

where the reference periods $T_B$ and $T_C$, that identify the limits of the constant pseudo-acceleration branch, are defined in Eurocode 8.

### 5.2 Estimation of the collapse-PGA by means of nonlinear dynamic analyses

An alternative and more reliable procedure for the evaluation of the collapse-PGA is based on the use of a sufficient number of artificially generated ground motions compatible with the prescribed pseudo-acceleration design spectrum.

By repeating the analyses iteratively, varying the peak ground acceleration, it is possible to identify the collapse-PGA value for each of the artificially generated ground motions compatible with the design spectrum. The most reliable estimation for the collapse-PGA can be identified with the mean value.

Due to the great number of nonlinear dynamic analyses required, this procedure involves a huge computational effort if applied to the three-dimensional model of the building, while it becomes relatively easy to apply if the dynamic analyses are performed on the nonlinear single-degree-of-freedom
equivalent system. It is obvious that this method still involves a much greater computational cost than the one previously described based on inelastic design spectra.

5.3 The seismic resistance of the considered building

The evaluation of the collapse-PGA of the considered building, according to the above described methods, yields to the results reported in figure 6a, for the transverse direction, and in figure 6b, for the longitudinal direction.

In each of the figures the bold-dashed line identifies the collapse-PGA value obtained by means of the Eurocode 8 inelastic design spectrum, while the marks indicate the collapse-PGA values estimated by means of nonlinear dynamic analyses conducted on the nonlinear single-degree-of-freedom equivalent system with reference to twelve artificial ground motions. The considered accelerographs have been generated through the computer software Simqke [17] to be compatible with the elastic design spectrum defined by Eurocode 8 for A-type soil. In each figure the mean value and a range of variation of one standard deviation are also reported. Moreover, the collapse-PGA values are summarised in table 7.

<table>
<thead>
<tr>
<th>Direction</th>
<th>EC8 inelastic design spectrum</th>
<th>Nonlinear dynamic analyses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse</td>
<td>0.150 g</td>
<td>0.213±0.022 g</td>
</tr>
<tr>
<td>Longitudinal</td>
<td>0.230 g</td>
<td>0.334±0.030 g</td>
</tr>
</tbody>
</table>

Figure 6: Collapse-PGA for the equivalent system. A-type soil. Excitation along the transverse (a) and the longitudinal (b) direction.

Table 7: Collapse-PGA for the equivalent system. A-type soil.
The obtained results confirm that the building is more vulnerable in the transverse direction. In fact the mean value of the collapse-PGA obtained by the nonlinear analyses in transverse direction, equal to 0.213 g, is about the 64% of the collapse-PGA in the longitudinal direction. The simpler methodology based on the inelastic design spectrum leads to lower values of the collapse peak ground accelerations, equal to 0.150 g in transverse direction and 0.230 g in the longitudinal direction. The difference between the two methods may be due to an underestimation of the effective ductility capacity of the single-degree-of-freedom equivalent systems that has been evaluated by means of a behaviour factor.

The obtained results seem to indicate that the building is not able to withstand the prescribed action for the construction site that, being classified as medium seismicity area, is associated to a peak ground acceleration equal to 0.25 g. However the building would be safe, with respect to the collapse condition, in low seismicity areas, which are characterised by a prescribed peak ground acceleration that may be assumed equal to 0.15 g.

6 Conclusions

In this first part the seismic resistance of a reinforced concrete building designed and constructed without seismic provisions has been estimated. The case-study building has been built in the late seventies in Eastern Sicily, a high seismic area where regulations have been enforced only in 1981. The problem of the evaluation of the seismic resistance of the existing building has been addressed by means of three-dimensional nonlinear analyses as well as of simplified procedures. The building has been modelled as a three-dimensional frame in which the nonlinear behaviour of the structure has been introduced by means of lumped plasticity elements. The performed nonlinear static analyses have also been used for the definition of equivalent nonlinear single-degree-of-freedom systems. Such simplified systems allow us to estimate the level of peak ground acceleration (PGA) which the building can withstand according to a prescribed design spectral shape.

The results have shown that the building would be nearly safe in low seismicity areas but would need seismic upgrading both in strong and moderate seismicity areas.

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and Environmental Engineering, University of Catania: Catania, Italy, 260 pages, 2003 (in Italian).


