Seismic vulnerability assessment of medium-rise buildings using the Index Method: the case of block #22 of Santa Maria Hospital

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Abstract

Following major seismic events occurred in densely populated areas, structural rehabilitation of buildings has become a major priority for local authorities in order to avoid severe structural damage and reduce casualties. Buildings located in high seismic hazard zones and demanding urgent structural rehabilitation spread all over the world. Therefore, quick assessment of reinforced concrete buildings seismic vulnerability became urgent on a large scale, establishing priorities to proceed with structural rehabilitation in order to fulfil the requirements of most recent design codes. The scope of this work is to present the Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings, developed by Japanese authorities, explaining its assumptions and theoretical bases. Also, the adjustment of that standard was also carried out to Portugal, reflecting local construction methods and detailing, taking into account EC8 concepts and the most recent release of the Portuguese National Annex. Furthermore, the applicability of such method to block #22 of the Santa Maria Hospital was tested, which had previously been evaluated using non-linear static (pushover) analysis, whose results were taken as reference.

Keywords: Seismic assessment; Seismic vulnerability; Hospitals; Hirosawa Method; Seismic index; Eurocode 8; National Annex

1. Introduction

1.1. Scope

Seismic rehabilitation became regarded as a top priority following significant damages to building structures as a consequence of major seismic events that occurred in densely populated urban areas, as were 1978 Miyagiken-oki, 1985 Mexico and 1989 Loma Prieta Earthquakes. Global awareness of the importance of earthquake countermeasures for existing vulnerable buildings was strongly pushed by 1994 Northridge and 1995 Hyogoken-Nanbu (Kobe) earthquakes.

Buildings located in earthquake prone areas, and in need of structural rehabilitation, spread all over the world. On the one hand, buildings damaged by past earthquake events should be repaired and strengthened (post-earthquake rehabilitation) in order to comply with present seismic design codes. On the other hand, seismically inadequate buildings, whose design does not comply with the requirements of current codes, may also need strengthening (pre-earthquake rehabilitation) so that their performance may satisfy current codes. Nowadays, the term rehabilitation is regarded, in the earthquake engineering field, as a general procedure that includes the concepts of repair, upgrading, retrofitting and strengthening, in order to reduce seismic vulnerability of existing buildings.

1.2. Importance of Health Facilities

Health facilities, as hospitals and health centres, have a fundamental role on providing assistance to populations in the event of a natural disaster, such as a strong earthquake. Disaster planning for these facilities is complex: they have high occupancy levels and house healthcare professionals, patients and visitors. For these reasons, it is essential that special consideration is given to assessing and reducing their vulnerability, so that structure does not collapse for different intensity seismic events and that equipments maintain their operability.

1.3. Damage on RC buildings following the Hyogoken-Nanbu Earthquake

Following the Hyogoken-Nanbu Earthquake, an exhaustive investigation of the damage on RC buildings was performed. Detailed results of the investigation can be found in [2-4]. A summarized version of the main results follows:

- Most of the buildings designed and built according to Japanese present seismic codes, whose latest revision dated from 1981, performed reasonably well, meaning that severe structural damage was prevented, as well as collapse for life safety.
- In areas that reported a seismic intensity equal or higher than 7 in the JMA\(^1\) scale, only 7.8% of buildings suffered severe damage or actually collapsed.

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\(^1\) JMA – Japan Meteorological Agency
The ratio of buildings that suffered severe damage or actually collapsed with soft first story reached 17%, but only 7.0% for those without soft first storey.

Damage to RC buildings was serious for those designed before 1981, particularly for those before 1971, as major revisions to Japanese seismic design code, which is from the 1924, were performed exactly in 1971 and 1981.

The ratio of seriously damaged and collapsed RC buildings designed before the 1971 code revision, 1981 code revision and since, are 8.1%, 3.7% and 1.1% for buildings without soft first storey, and 12.2%, 11.7% and 2.4% for buildings with soft storey, respectively.

Therefore, the seismic performance assessment of a large number of buildings which had never experienced severe ground motion before became an evident need, in order to identify those that stand as most vulnerable. Those identified as most vulnerable should be rehabilitated, whichever technique applies to them, in order to fulfil with actual seismic code performance requirements.

2. Index Method

2.1. General procedure and index computation

The Standard for Seismic Evaluation of Existing RC Buildings [6], published by the Japan Building Disaster Prevention Association, is based on a method developed by M. Hirosawa [7]. The method was originally proposed for vulnerability assessment of existing or damaged buildings of up to 8 floors, consisting of RC frames and/or wall structures. Structural vulnerability assessment is performed through comparison of $I_s$ (seismic index of structure) with $I_{s0}$ (seismic demand index).

The seismic capacity of a building is measured by the $I_s$ index that should be computed at each storey and in both main directions. $I_s$ index is defined by the following equation:

$$I_s = E_0 \times S_D \times T$$  \hspace{1cm} (1)

where $E_0$ is the basic seismic index, $S_D$ is the structural irregularity index and $T$ is the time index. $S_D$ index accounts for the effects of shape complexity and stiffness unbalanced distribution on the overall seismic performance of the structure. $T$ index evaluates the effects of structural defects such as cracking, deflection, aging, along others, on the structural performance.

Basic seismic index $E_0$ is evaluated as a function of the strength index, $C$, and the ductility index $F$:

$$E_0 \alpha C \times F$$  \hspace{1cm} (2)

The Index Method consists of three levels of screening procedures for $I_s$ index computation. On the first level of screening, global shearing strength is estimated for each floor and direction, as failure of structural elements by flexural yielding is neglected. $S_D$ and $T$ indexes computation is quite simple and based on visual inspections only. The second level requires the calculation of the ultimate limit strength of columns and walls considering reinforcing bars detailing. Finally, in the third level of screening, the ultimate strength of columns, walls and beams is required, as an early failure of beams connecting vertical load bearing elements is taken into account. $S_D$ and $T$ indexes computation is quite more complex on second and third levels of screening.

The seismic demand index ($I_{s0}$) is defined by the following equation:

$$I_{s0} = E_S \times Z \times G \times U$$  \hspace{1cm} (3)

where $E_S$ is the basic seismic index of the structure, $Z$ is the zone index, $G$ is the ground index and $U$ the usage index. $Z$ index accounts for the seismic activities and seismic intensities expected in the region of the site. $G$ index takes into consideration the amplification effects of soil surface, geological conditions and soil-structure interaction on the earthquake motions. $U$ index accounts for the buildings importance according to its use. Basic seismic index of structure, $E_S$, is selected regardless of the direction or...
storey under assessment and is assumed 0.8, for first level screening, and 0.6, for second and third levels of screening procedure. Values of $E_S$ given by the Standard for Seismic Evaluation of Existing RC Buildings [6] were determined based on the study of building damage, following the Tokachi-oki (1968) and Miyagiken-oki (1978) earthquakes, that compared visual assessment of building damage with $I_S$ index computation. Figure 3 shows that $I_S$ index properly distinguishes damaged buildings from non-damaged buildings. Peak ground accelerations (PGA) of the mentioned earthquakes were estimated as 250 Gals.

Similar study was carried out for RC school buildings that suffered the 1995 Hyogoken-Nanbu Earthquake and concluded that the value 0.6 for the $I_S$ index is the border line between severe and moderate damage [10].

2.2. Basic concept of the Index method

The seismic performance of RC buildings is quite variable depending on the combination of ductility and strength that the buildings possess. The $E_0$ index evaluates the seismic performance of the building based on its ductility and strength. Example buildings A and B are cited here to serve as an explanatory example. Building A is considerably strong but low in ductility, as it is assumed that it has many walls. On the other hand, Building B is not so strong but possesses large ductility, as it is assumed to be a rigid-frame structure. The following figure shows the relationship between horizontal force and horizontal displacement when the force is acting on the RC buildings.

When earthquake loading is acting on the building, the building remains safe if the maximum displacement indicated by the mark ▼ remains within the failure point shown by the x mark. If not, severe damage will occur.

From different studies, it is known that, in order to verify safety to seismic loading, RC buildings with many walls should possess considerable strength, as they are short on ductility, and that rigid-frame buildings should have considerable ductility, as they are not so strong. Based on this principle, the $E_0$ index is introduced in order to establish evaluation criteria usable for the common buildings above mentioned. Example Buildings A and B are very simple ones but actual buildings are never so simple or so complicated. Figure 5 is a schematic description of a rigid-frame building behaviour with limited amount of walls when applied by a gradually increasing horizontal force.

Figure 3 – Index $I_S$ and building damage following Tokachi-oki and Miyagiken-oki earthquakes [9]

Figure 4 – Relationship between horizontal force and horizontal displacement of RC buildings [6]

Figure 5 – Behaviour of combined rigid-frame and wall buildings [6]

When the walls reach failure at point (a), the building does not collapse. Although horizontal forces drop for a moment due to the loss of rigidity of the building, rigid-frame begins to resist horizontal forces as deformation increases. Building collapses at point (b) when the rigid-frame structure reaches its failure.

If a storey could be idealized as a series of vertical elements, as shown on Figure 6 (top), the load-deflection relationship for each storey could be represented by curves also on Figure 6 (bottom). The basic concept behind $E_0$

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2 Gal (from Galileo) is the acceleration unit in the CGS system (centimeter, gram, second) (1Gal = 0.01 ms$^{-2}$)
index formulation is to evaluate the energy dissipation capacity of the storey under analysis.

Figure 6 – Element ideal load-displacement relationship [1] (α₁ to α₃ parameters are explained below)

2.3. First level screening

The first level procedure is the simplest of the three levels but is useful for buildings with considerable amount of walls. E₀ index represents the absolute shearing strength of each floor. For the calculation of E₀ index, all elements that form part of the seismic resistant structure must be classified in one of the following categories:

- RC columns – all the columns with h₀/D ratio greater than 2, being h₀ the vertical clearance and D the width of the cross-section;
- RC short columns – all the columns with h₀/D ratio smaller than 2. The seismic behaviour of these columns is controlled by shear failure (brittle) and characterized by low capacity of inelastic deformation (ductility);
- RC walls – reinforced concrete elements in which the ratio between the wider and thinner sides of the cross-section is greater than 3.

In order to gather the required information to proceed with element categorization, an investigation of the building should be conducted on the following items [6], which are necessary to derive the seismic index of the structure (Iₛ) for first level screening:

- Material strengths and cross-sectional dimensions for calculation of structural members strengths;
- Cracking in concrete and deformations of structure for evaluation of time index T;
- Building configuration for evaluation of irregularity index Sᵢᵦ. Seismic capacity must be calculated first by considering the failure of the weakest elements. Nevertheless, if the failure of this group does not produce instability in the system, seismic capacity must be calculated by considering the next group and not considering the resistance of elements that have reached failure. Eₛ index is defined by the following equation:

\[
E_0 = \phi \times (\alpha_1 \times C_{SC} + \alpha_2 \times C_W + \alpha_3 \times C_C) \times F \quad (4)
\]

where C_SC, C_W and C_C are the resistance indexes of short-columns, walls, and columns, respectively. F index is the ductility index associated with each failure type considered on calculation: type A, B or C.

Type A failure is controlled by short columns failure, which means that their total shearing capacity is considered (α₁=1.0), but only a portion of the shearing capacity of walls and columns (α₂=0.7; α₃=0.5), as shown in Figure 7.

Type C failure is controlled by the columns, meaning that both short-columns and walls have already collapsed and, as a consequence, their contribution to resistance is neglected (α₁=α₂=0.0).

Type B failure is an intermediate situation where failure is controlled by the walls; a portion of the shearing capacity of columns is considered but short columns contribution is neglected as they have already collapsed.

The resistance indexes (C_SC, C_W and C_C) are computed by the sum of the product of the area of the cross-section of a wall or column (Aᵢ) by the average shear stress at ultimate limit state (τᵢ). Furthermore, this product is then reduced by αᵢ coefficients that account for the fact that some elements may reach their failure for an inter-storey drift smaller than others (e.g., short columns and masonry walls when compared with RC columns and walls), as seen on Equation (4).
As the number of storeys considered increases, its value decreases as the number of storeys, a few remarks can be made:
- In spite of the number of storeys, its value is always 1.0 at storey level 1;
- For a given storey number, its value decreases as the number of the storey under assessment increases;
- Its value at the top storey decreases as the considered total number of storeys of the building is bigger.
- As the number of storeys considered increases, its value at the top storey converges to a minimum of 0.5.

Detailed formulas and coefficient values used for calculation of each index can be found on the Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings. [6]

2.4. Shear modification factor $\phi$

The shear modification factor is responsible for the vertical distribution of seismic shear forces at each storey level. The theoretical derivation performed for the shear modification factor ($\phi$) allowed to identify the basic assumptions behind its expression. Initially, a linear deflection for the fundamental mode was assumed and that base shear force was given by the product of the building’s total weight by a seismic coefficient ($\beta$). The general expression presented by the Index Method for $\phi$ was later obtained by considering a uniform floor weight distribution and constant inter-storey height. The complete derivation of the $\phi$ factor is in Derivation 1.

By computing the expression of the $\phi$ factor for different number of storeys, a few remarks can be made:
- In spite of the number of storeys, its value is always 1.0 at storey level 1;
- For a given storey number, its value decreases as the number of the storey under assessment increases;
- Its value at the top storey decreases as the considered total number of storeys of the building is bigger.
- As the number of storeys considered increases, its value at the top storey converges to a minimum of 0.5.

2.5. Similar Method – PAHO

The document entitled Principles of Disaster Mitigation in Health Facilities [2] published by PAHO focuses on the impact that natural disasters have in such important and critical facilities as hospitals. It introduces various aspects of vulnerability assessment and addresses the application of practical measures in order to mitigate damage in hospitals on a structural and non-structural level, as well as administrative and of internal organization.

In its Annex, screening level 1 of the Index Method is briefly introduced and with two significant refinements in $E_0$ index calculation:

- RC walls are categorized into four classes according to reinforcement adopted and failure type, considering different average shear stress at ultimate level for each of them;
- Infilled brick walls contribution to strength is considered. These walls include unreinforced brick panels located between frame columns.
- Reinforced brick walls contribution to strength is considered. These walls include reinforced brick panels or brick panels confined with thin elements of reinforced concrete.

The above mentioned walls regard those that have been designed and built in order to transmit horizontal and vertical loads to lower levels and foundation. Walls that only resist their own weight, like partition walls that are...
isolated from the seismic-resistant structure, are not
considered. \( E_0 \) index calculation expression follows:
\[
E_0 = \phi \times \left[ \alpha_1 \times (C_{ma} + C_a + C_{sc}) + \alpha_2 \times C_a + \alpha_3 \times C_{sc} \right] . \]
where \( C_{ma}, C_a \) and \( C_{sc} \) are the resistance indexes exhibited
by the infilled brick walls, the unreinforced or partially
confined brick walls and the confined brick walls.

3. National Portuguese Variant of the Index Method

3.1. Introduction

The \( I_S \) index is responsible for estimating the storey
shearing capacity at each storey of the building. Firstly, that
capacity is calculated for each structural element, as the
product of an average shear stress at ultimate limit state \( (\tau_i) \)
by its cross sectional area \( (A_i) \), as shown on the expressions
presented on Derivation 2. Then, these individual element
capacities are combined by effective strength factors \( (\alpha_i) \),
as shown on expression (4).

The values of \( \tau_i \) and \( \alpha_i \) are the result of several
experimental tests, reflect standard Japanese structural
solutions, materials used, common reinforcement detailing
and were certainly submitted to calibration. Their straight-
forward applicability to Portuguese reality must be
checked.

As a consequence of the substantial difference of
seismic activity between Portugal and Japan and the ad-hoc
values proposed for the \( I_{S0} \) index by the Standard for
Seismic Evaluation of Existing Reinforced Concrete Buildings [6], an expression for \( I_{S0} \) index calculation is
proposed.

3.2. Seismic Index of the Structure (\( I_S \))

In 1990, Felicita Pires tested several rigid-frame model
structures infilled with brick panels consisting of two
columns and a beam. Those tests consisted in imposing
increasing horizontal displacements at the top of the
structure (beam level) by applying horizontal forces;
meanwhile, columns were compression stressed to recreate
the effect of upper floors average loading of a medium rise-
building. That compression stress reached an average of 4.5
to 5.0 MPa during testing. The remaining 6 infilled frame
models differed in terms of: stirrups spacing, longitudinal
reinforcement adopted, anchorage lengths and the
construction process of the infilled brick panels. Materials
used in the experimental test were C15/20 concrete
\( f_{cd}=10.7 \text{ MPa} \), reinforcing steel bars of class A400
\( f_{cd}=348 \text{ MPa} \) and bricks with dimensions
30x20x15 [cm].

The way the Index Method is computed does not allow
for a direct comparison with these experimental testing
results as \( I_{S0} \) index is adimensional. So, Index Method
expressions were rearranged in order to make that
comparison possible (see Derivation 2).

Model 1 consisted in the base frame model. Calculation of
the shearing capacity estimated by the Index Method follows:
• Compressive strength of concrete: \( f_{cd}=10.7 \text{ MPa} \) ⇒
  \( \beta_c = 10.7 / 20 = 0.535 \);
• Column categorization: \( h_0/D = 1.625/0.15 = 10.9 > 6 \) ⇒
  \( \text{Column Type C2} \Rightarrow \tau_c = 0.7 \text{ MPa} \);
• Ductility index (Type C - ductile failure) \( F = 1.0 \);
• Shearing capacity: \( V_{cd} = A_c \times \tau_c \times \beta \times F \) ⇒
  \( V_{cd} = 0.15^2 \times 2 \times 700 \times 0.535 \times 1.0 = 16.85 \text{ kN} \).

<table>
<thead>
<tr>
<th>Model</th>
<th>( M_{\text{max}} ) [kN]</th>
<th>( d_{\text{col}} ) [mm]</th>
<th>( \delta_c/H )</th>
<th>( f_{cd} ) [MPa]</th>
<th>( c_{\text{max}} ) [MPa]</th>
<th>( V_{\text{max}} ) [kN]</th>
<th>( d_{\text{col}} ) [mm]</th>
<th>( \delta_c/H )</th>
<th>( f_{cd} ) [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1</td>
<td>-39</td>
<td>-41</td>
<td>1/40</td>
<td>0.85</td>
<td>-31</td>
<td>-100</td>
<td>1/17</td>
<td>0.68</td>
<td></td>
</tr>
</tbody>
</table>

Model M1 testing results, are presented on Table 1. Comparison of the final results leads to the conclusion that
the calibration of the Index Method underestimates by 50%
the shearing capacity of the rigid-frame structure on Type
C failure mode.

The \( \phi = \frac{\delta_c}{H} \) ratio\(^3\) observed at maximum capacity is about
1/40, which is greater than that assumed for a Type C
failure mode that is generally taken as 1/150, as shown on

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3 \( \delta_c \) – Horizontal displacement; \( H \) – frame structure height.
Figure 6. This fact could be one of the possible reasons for such a difference between tested and computed results.

Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings [6] does not consider the contribution to shearing capacity by masonry walls. Therefore, expressions presented by PAHO [2] were used to derive the capacity of the remaining 6 models (infilled frame models):

- Ductility index (Type C - brittle failure) ⇒ \( F = 0.8 \);
- Average shear stress at ultimate limit state of masonry:
  \[ \tau_{\text{mar}} = 0.6 \times 0.85 \times 0.5 = 0.255 \text{ MPa} \]
- Shearing capacity:
  \[ V_{\text{rd}} = (2.10 \times 0.15 \times 255 \times 0.5) \times 0.8 = 71.0 \text{ kN} \]

All models showed a shearing capacity higher than the value obtained by the Index Method computation. On Models M2, M3 and M6, the frame structure was infilled only with the brick walls (after concreting of the beam and columns), which is common practice in Portugal. Results show an underestimation of the shearing capacity that varies between 34% (M3) and 54% (M6). Model M2, that presents lower anchorage length of beam reinforcement and tighter stirrups spacing on the beam-column nodes, showed a 41% deviation from computed results. This fact reiterates the importance of these details on global performance of framed structures. The \( \delta_b/H \) ratio observed at maximum capacity, on Models M2, M3 and M6, is about 1/500, which is similar to the value assumed for Type A failure mode, as shown on Figure 6. Test results from Models M2 to M7, obtained by Felicita Pires, are presented on Table 2.

### Table 2 – Test results from Models M2 to M7. [11]

<table>
<thead>
<tr>
<th>Model</th>
<th>At maximum capacity</th>
<th>On model collapse</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( F_{\text{max}} ) [kN]</td>
<td>( d_{\text{max}} ) [mm]</td>
</tr>
<tr>
<td>M2</td>
<td>100</td>
<td>1.6</td>
</tr>
<tr>
<td>M3</td>
<td>108</td>
<td>1.9</td>
</tr>
<tr>
<td>M4</td>
<td>109</td>
<td>1.2</td>
</tr>
<tr>
<td>M5</td>
<td>109</td>
<td>1.2</td>
</tr>
<tr>
<td>M6</td>
<td>154</td>
<td>2.8</td>
</tr>
<tr>
<td>M7</td>
<td>140</td>
<td>2.8</td>
</tr>
</tbody>
</table>

Final results for each failure mode (Type A and C) show that the calibration of the Index Method, presented by the Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings [6], consistently underestimates values obtained by experimental tests performed by Felicita Pires. Underestimation is positive for a simplified method; such conservativeness may lead to difficulties in the application on the Index Method to national territory. Therefore, an adequate calibration of \( \tau_r \), \( \alpha_i \) and \( F \) parameters may be required.

3.3. Seismic Demand Index of the Structure (I\(_S\))

General concepts and parameters defined in EC8 take part of the expression proposed for I\(_S\) index computation, enabling it to reflect the future assessment of the seismic action proposed by EC8. A thorough theoretical derivation follows.

Considering that I\(_S\) value corresponds to the seismic coefficient, taken as the ratio between base shear force and weight,

\[ I_{S} = \beta_{x} \]

The seismic coefficient \( n \) is given by:

\[ \beta_{x} = \frac{R_{x}}{W_{x}} \]

Where \( R_{x} \) is the base shear force for a given mode \( x \) along \( x \) direction and \( W_{x} \) is the weight of the superstructure. \( W_{x} \) is related to the mass matrix (M) and the \( \mathbf{1}_{x} \) vector as shown next:

\[ W_{x} = g \cdot M_{x} = g \cdot \mathbf{1}_{x}^{T} \cdot M \cdot \mathbf{1}_{x} \]

Base shear force can be expressed as a function of the inertial forces vector \( F_{in} \) of mode \( n \) by:

\[ R_{x} = \mathbf{1}_{x}^{T} \cdot F_{in} \]

The inertial forces vector may be expressed as a function of the modal shape vector of mode \( n \) (\( \phi_{n} \)) and the corresponding values of the modal participation factor \( P_{x} \) and spectral acceleration \( S_{x} \) as:

\[ F_{in} = M \cdot \phi_{n} \cdot S_{x} \cdot P_{x} \]

Modal participation factor may be derived by:

\[ P_{x} = \phi_{x}^{T} \cdot M \cdot \mathbf{1}_{x} \]

Which may be related with the total mass of the building \( M_{x} \) by \( \lambda_{x} \) as shown next:

\[ \lambda_{x} = \frac{\mathbf{1}_{x}^{T} \cdot F_{\text{max}}}{M_{x}} \]

Where,

\[ M_{x} = \mathbf{1}_{x}^{T} \cdot M \cdot \mathbf{1}_{x} \]

Rewriting expression (9),

\[ R_{x} = \mathbf{1}_{x}^{T} \cdot M \cdot \phi_{n} \cdot S_{x} \cdot P_{x} \]

Where \( S_{x} \) is the spectral acceleration of mode \( n \). Expression (14) may be rewritten as:
\[ R_{xn} = \varphi_n^T \cdot M \cdot 1_k \cdot S_{an} \cdot P_{xn} \]  \hspace{1cm} (15)

Or, by considering equation (11),
\[ R_{xn} = S_{an} \cdot P_{xn}^2 \]  \hspace{1cm} (16)

Then, the seismic coefficient may be given by:
\[ \beta_{xn} = \frac{R_{xn}}{W_x} = \frac{S_{an} \cdot M_x \cdot \lambda_{xn}}{g \cdot M_x} \]  \hspace{1cm} (17)

This means,
\[ \beta_{xn} = \frac{R_{xn}}{W_x} = \frac{S_{an} \cdot \lambda_{xn}}{g} \]  \hspace{1cm} (18)

For the fundamental mode along x direction, assuming \( n=1 \), and considering the design response spectrum for elastic analysis (EC8, 3.2.2.5), the seismic coefficient would be given by:
\[ \beta_{x1} = \frac{S_d(T_1) \cdot \lambda_{x1}}{g} \]  \hspace{1cm} (19)

Where \( S_d(T_1) \) is the design spectrum ordinate of the fundamental mode along x direction. Considering that the period of fundamental mode is greater than \( T_B \) (lower bound of the constant acceleration branch),
\[ \beta_{x1} = \frac{S_e(T_1) \cdot \lambda_{x1}}{g} \]  \hspace{1cm} (20)

Where \( S_e(T_1) \) is derived from the elastic response spectrum and \( q \) is the behaviour factor for elastic design. Final seismic coefficient \( \beta_x \) may be obtained by modal combination (CQC for example):
\[ \beta_x = \sqrt{\sum_{n=1}^{N} \beta_{xn}^2} = \frac{1}{g \cdot q} \sqrt{\sum_{n=1}^{N} S_e(T_n) \cdot \lambda_{xn}} \]  \hspace{1cm} (21)

From the previous expression may be concluded that, for ordinary buildings, the contribution of the fundamental mode is governing for seismic coefficient assessment. As a matter of fact, the mass participation ratio is close to 1.0 and that fact is enhanced by the mentioned modal combination procedure. In such circumstances, the final seismic coefficient equals, approximately, that given by the sole contribution of the fundamental mode:
\[ \beta_x \cong \beta_{x1} = \frac{S_e(T_1) \cdot \lambda_{x1}}{g \cdot q} \]  \hspace{1cm} (22)

As a conservative approach, the previous equation may be rewritten considering that the fundamental mode period lies within the constant acceleration branch of the spectrum.

Finally, neglecting the contribution of all mode shapes other than the fundamental mode and assuming that the fundamental period is such that acceleration is maximum (constant acceleration branch of the response spectrum), \( I_{S0} \) can be expressed by:
\[ I_{S0} = \frac{2.5 \cdot a_{gr} \cdot S \cdot \lambda_{x1} \cdot \gamma_1}{g \cdot q} \]  \hspace{1cm} (23)

where,
- \( a_{gr} \) is the peak ground acceleration for ground type A in a given seismic zone (National Annex value definition);
- \( S \) is the soil factor that reflects local geotechnical conditions;
- \( \gamma_1 \) is the importance factor according to the importance class of the building;
- \( g \) is gravitational constant (9.8 ms\(^{-2}\));
- \( \lambda \) is the correction factor that accounts for the fact the effective modal mass of the 1\(^{st} \) mode is smaller than the total mass of the building; [12]
- \( q \) is the behaviour factor which is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic, to the seismic forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure. [12]

### 3.4. Correction factor \( \lambda \)

Effective modal mass is automatically calculated when performing dynamic analysis but not when applying simplified static methods to assess base shear force. EC8 introduces a correction factor to account for that issue when using the Lateral Force Method of Analysis.

[Figure 9 – Effective modal mass for the 1\(^{st} \) mode [%].](image)

In buildings with at least three storeys, effective modal mass of the 1\(^{st} \) (fundamental) mode is smaller, on average 15\%, than the total mass of the building [12]. Theoretical derivation of the effective modal mass of the 1\(^{st} \) mode has been performed for a simple structure model assuming concentrated masses at each storey and constant storey height, similar to the one shown on Figure 8.
Modal shape column vector and mass matrix of the fundamental mode:

\[
\begin{bmatrix}
1/n \\
2/n \\
\vdots \\
 n/n
\end{bmatrix}; \quad [M]= \begin{bmatrix}
m & 0 & \ldots & 0 \\
0 & m & \ldots & 0 \\
\vdots & \vdots & \ddots & \vdots \\
0 & 0 & \ldots & m
\end{bmatrix}; \quad \phi_i = \frac{v_i}{\sqrt{v_i^T \cdot M \cdot v_i}} = \frac{1}{\sqrt{m \times \sum_{i=1}^{n} \left( \frac{1}{i} \right)^2}} \times \begin{bmatrix}
1/n \\
2/n \\
\vdots \\
 n/n
\end{bmatrix}
\]

V_{1T} \cdot M = \begin{bmatrix}
m & 2m & 3m & \ldots & Nm \\
2m & 3m & \ldots & Nm \\
\vdots & \vdots & \ddots & \vdots \\
Nm & Nm & \ldots & Nm
\end{bmatrix};

Assuming that the fundamental mode shape is linear in height, the modal configuration column vector and mass matrix are expressed as shown on Derivation 3. The rest of the derivation follows. Figure 9 shows the effective modal mass for the 1st mode, computed according to the expressions on Derivation 2, for buildings with ten or less storeys. Graphic observation shows that effective modal mass is approximately 85% for a 3-storey building, and increasingly less for higher buildings, as expected.

3.5. Portuguese National Annex to EC8

For the purpose of EC8, national territories are subdivided by National Authorities into seismic zones, depending on local seismic hazard. The values used for seismic action are classified as Nationally Determined Parameters, therefore defined in the National Annex.

The following presentation of the National Annex document is based on an article published in 2007 by Cansado Carvalho entitled “Consequences to seismic design in Portugal” [13], as well as the preliminary version of the National Annex [14].

Two seismic action scenarios have to be considered in national territory: Type 1 (interplate earthquake) and Type 2 (intraplate earthquake). In Madeira, only Type 1 applies, while in Azores, only Type 2 applies. Seismic zoning depends on the seismic scenario considered.

Table 3 – Peak ground acceleration for seismic zones. [14]

<table>
<thead>
<tr>
<th>Type 1 (intraplate)</th>
<th>Type 2 (interplate)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic Zone</td>
<td>a_{g0} [m/s^2]</td>
</tr>
<tr>
<td>1.1</td>
<td>2.5</td>
</tr>
<tr>
<td>1.2</td>
<td>2.0</td>
</tr>
<tr>
<td>13</td>
<td>1.5</td>
</tr>
<tr>
<td>14</td>
<td>1.0</td>
</tr>
<tr>
<td>1.5</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Peak ground acceleration for each seismic zone and scenario are represented in Table 3.

The shape of the elastic response spectrum, associated with each seismic action type, is defined by values of the periods T_{B}, T_{C}, and soil factor S. These values depend upon ground type. Table 4 shows the values of the periods T_{B}, T_{C}, and T_{D} for ground type A (rock). Values in brackets are recommended by EC8 and are shown for comparison only.

Soil-structure interaction can cause amplification of the effects of seismic action on the structure. Ground types A, B, C, D, and E may be used to account for the influence of local ground conditions on the seismic action. Tables 4, 5 and 6 present values of the previously mentioned Nationally Determined Parameters, used to define each seismic scenario.

Table 4 – Soil factor and periods T_{B}, T_{C} and T_{D} values. [14]

<table>
<thead>
<tr>
<th>Factor</th>
<th>Type 1</th>
<th>Type 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>S</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>T_{B} (s)</td>
<td>0.1 (0.15)</td>
<td>0.1 (0.05)</td>
</tr>
<tr>
<td>T_{C} (s)</td>
<td>0.6 (0.4)</td>
<td>0.25</td>
</tr>
<tr>
<td>T_{D} (s)</td>
<td>2.0</td>
<td>2.0 (1.2)</td>
</tr>
</tbody>
</table>

Table 5 – Soil factor and period T_{C} values for Scenario Type 1. [14]

<table>
<thead>
<tr>
<th>Seismic Scenario Type 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground Type</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>S</td>
</tr>
<tr>
<td>A</td>
</tr>
<tr>
<td>B</td>
</tr>
<tr>
<td>C</td>
</tr>
<tr>
<td>D</td>
</tr>
<tr>
<td>E</td>
</tr>
</tbody>
</table>
Buildings are classified into 4 importance classes, depending on the consequences of collapse for human life, on their importance for public safety, as well as civil protection in the immediate post-earthquake period, and on the social and economic consequences of collapse [12]. The importance classes are characterized by different importance factors $\gamma_I$ as described in Table 7.

The different levels of reliability are obtained by multiplying the reference seismic action or, when using linear analysis, the corresponding action effects by this importance factor. The design ground acceleration on type A ground $a_g$ is equal to $a_{gR}$ times the importance factor $\gamma_I$, as shown by the expression (7).

$$a_g = \gamma_I \times a_{gR} \quad (24)$$

### 4. Seismic Vulnerability Assessment of block #22 of the Santa Maria Hospital

#### 4.1. Santa Maria Hospital overview

The Santa Maria Hospital, hereafter referred as SMH, has a total area of 120,000 m$^2$, presently with 1560 internment...
beds. The hospital complex is located in Lisbon. The SMH building consists of 47 cast-in-situ reinforced concrete frame building blocks, 1 to 11 storeys high, separated by expansion joints. The building design and construction started in the late 1930s and ended in 1953, before earthquake-resistant design clauses were included in the Portuguese structural design codes. Nevertheless, the structural designer was aware of the seismic hazard, but considered that the RC frame structure had an intrinsic lateral force resisting strength that would be sufficient in the event of an earthquake.

4.2. Santa Maria Hospital structure description

The SMH presents two main, E-W oriented wings, with three N-S direction connection building blocks as shown in Figure 2.

The majority of the building blocks are 10-storeys high. The prevailing structural solution consists of cast-in-situ 2D RC frames with cast-in-situ ribbed slabs (the ribs being set perpendicular to the frame beams). The building blocks were originally designed for intense wind pressures on the exposed façades, leading to the RC frames being predominantly set perpendicular to these façades. The building blocks in the two main wings have their frames in the N-S direction, with the exception of the corner blocks that present frames in both horizontal directions. The N-S direction connection blocks have their frames in the E-W direction.

In spite of not having been considered as a part of the structural load-resisting system (lateral or vertical), the building blocks have structurally non-negligible internal and external masonry walls. These masonry walls contain rubble stone blocks in the lower storeys and dense ceramic bricks in the upper storeys.

The reinforced concrete element detailing is characterized by low ductility, as can be expected by the use of smooth reinforcing bars, large stirrup spacing in the columns and outdated detailing rules.

The building blocks were designed for a uniformly distributed wind pressure of 1.47 kN/m$^2$ applied on the exposed façades.

The consulted design documentation had no reference to material strengths, either steel or concrete. Inverse design calculations point to the smooth steel rebars being of class S400 (characteristic yield stress of 400 MPa) [17]. The lack of information regarding the most relevant mechanical properties of both steel and concrete led to an experimental material characterization program. The average concrete compressive strength, as determined from a group of drilled concrete-core samples, was 20 MPa for beam and column elements. The steel reinforcement bars yielding stress varied, with an average value of 307 MPa.

4.3. Building block #22 of SMH

The building block #22 is located in the north wing of the Hospital, as shown in Figure 2. The structure consists of nine storeys, with six N-S, reinforced concrete plane frames with a spacing of approximately 5.75 m. The building plan is rectangular, 28.91x12.65 m for the first 5 floors, with a setback of 4.31 m in the direction of the frames for the top storey. The storey height varies between 3.0 and 4.0 m. The cross-sectional dimensions of members vary along the height, particularly the columns whose cross-sectional area at the top floor is less than 25% of that of the lowest storey. All the slabs are one-way, spanning between the frames (Figure 11).

4.4. Pushover analysis of block #22 of SMH

Non-linear static (pushover) analyses were performed on building block #22 of SMH [18] by João Almeida. Final results registered are crucial for comparison with the Index Method’s results (screening level 1).

The building is located in the Portuguese highest seismic risk zone, with a PGA value of 0.275g for a return period of 3000 years. Recent earthquake hazard studies [16] pointed out that this PGA value corresponds to a return period of 975 years. The structure was modelled using a 3D finite element model, with frame elements representing beams and columns. Considering that the analysis was performed in the N-S direction (direction of the expansion joints), no interaction with adjacent building was accounted for. The external and internal masonry infill panels were also modelled using a pair of diagonal frame elements for each panel, as shown on Figure 13. The storey slabs were modelled as rigid diaphragms. The non-linear static analysis requires the consideration of the actual force-deformation relationships for all sections which, in this case, were based on the longitudinal reinforcements of the
beams and columns, taken from the original project drawings. The numerical analyses (dynamic elastic and static inelastic) were carried out with SAP2000 [19].

The lateral load pattern applied along the building height was based on the elastic fundamental mode. The increments of this lateral load pattern leads to a capacity in terms of the normalized base shear versus drift. This curve shows the yielding, cracking and crushing sequence of the various elements. The top three storeys were the least affected, while the lower ones presented generalized degradation. For increasing top displacement values, damage tended to concentrate at the intermediate levels (2nd, 3rd and 4th storeys), and, eventually, all struts at the 3rd storey reached their ultimate strength. Further loading increments resulted in the development of the well-known soft-storey phenomenon. Figure 13 (a) shows the damage inflicted both in the masonry and in the reinforced concrete elements, at this step. More masonry elements at other levels have also reached their ultimate capacity at the onset of the soft-storey mechanism. The damage in the RC elements is limited to yielding in some beams, especially those at the second and third levels. At certain stage in the lateral loading increments, it is likely that an element (or group of elements) starts to significantly lose its (their) strength. It is convenient to reduce or eliminate the stiffness of the damaged element (or group of elements) and create the additional capacity curves to an accurate global structural characterization.

Model 1 is abandoned and Model 2 is created taking into account the elements that have failed. The final composite curve was assembled from these different capacity curves. The base shear and top displacement values registered at the performance point (PP) are, respectively, 7980.0 kN and 0.1153 m. The final base shear is composed of 6007.1 kN of Model 1 and only 1672.9 kN of Model 2. It should be noted that the equivalent viscous damping is 13.88%, for a period that is lengthened from an initial value of 0.488 s to 1.153 s.

A detailed analysis of the stress increment and deformation development in the masonry elements of Model 2 was carried out. The main conclusion was that the observed stress increments in the truss elements, when added to the stress levels resulting from Model 1, did not lead to the formation of another soft storey. Plastic hinge formation at PP can be observed in Figure 13(b), which was assembled from results of Model 1 (for concrete elements outside the soft-storey and masonry) and Model 2 (for concrete elements adjacent to the soft-storey).

![Figure 12 – Final Capacity Curve [17]](image)

![Figure 13 – (a) Layout of the damage at the instant of the soft-storey formation; (b) Expected damage at performance point. [18]](image)

The results obtained in building block # 22 have shown that, although collapse is not imminent, there is a large possibility of extreme damage concentration between two particular storeys, leading to high localized inter-storey drift demand that could undermine the Hospital’s functionality. As can be seen from Figure 13(b), although the risk of life-threatening injury is not negligible, some margin against total or partial structural collapse remains.

4.5. Assessment of block#22 of HSM with the Index Method

In order to perform the comparison of João Almeida’s pushover analysis with the Index Method, at the moment of the soft-storey mechanism formation, parameter values used to determine seismic action on the building should be
similar. The differences between both methods of analysis have to be taken in consideration.

4.5.1. Seismic action

Shear forces distribution at each level, at the moment of the soft-storey mechanism formation, registered by the pushover analysis are presented in Figure 14. The value of the base shear force was 19396 kN.

As previously stated, the seismic index $I_{S0}$ value is a seismic coefficient. Taking into account that the total mass of the building is 4687 tons, the derived value of index is 0.414.

4.5.2. Seismic capacity

Categorization of structural elements was based on element geometry. The ratio $h_0/D$ used for columns, showed that the columns of the first two storeys were C1’s and the rest C2’s. First two storeys are the shortest in height, with 2.90m and 3.30m, respectively, and columns at these levels have the largest cross-section areas of the building, thus resulting in an $h_0/D$ ratio smaller than 6.

<table>
<thead>
<tr>
<th>Storey</th>
<th>$A_i$ [m$^2$]</th>
<th>$V_{c}$ [kN]</th>
<th>$V_{mar}$ [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12.82</td>
<td>12824</td>
<td>15144</td>
</tr>
<tr>
<td>2</td>
<td>9.38</td>
<td>9380</td>
<td>14474</td>
</tr>
<tr>
<td>3</td>
<td>8.15</td>
<td>5706</td>
<td>13048</td>
</tr>
<tr>
<td>4</td>
<td>6.58</td>
<td>4604</td>
<td>13353</td>
</tr>
<tr>
<td>5</td>
<td>6.11</td>
<td>4277</td>
<td>13492</td>
</tr>
<tr>
<td>6</td>
<td>4.50</td>
<td>3150</td>
<td>9243</td>
</tr>
<tr>
<td>7</td>
<td>3.86</td>
<td>2701</td>
<td>9243</td>
</tr>
<tr>
<td>8</td>
<td>2.98</td>
<td>2083</td>
<td>9243</td>
</tr>
<tr>
<td>9</td>
<td>2.53</td>
<td>1772</td>
<td>9243</td>
</tr>
</tbody>
</table>

$I_S$ index was computed according to the Index Method expressions, except for the shear modification factor $\phi$ which had to derived according to expression (24), which accounts for the inertial forces distribution in height registered in the pushover analysis.

$$\psi = \frac{V_i}{W_i}$$

(24)

where, $V_i$ is the shear force registered at level $i$ and $W_i$ is the weight of the building above story level $i$. $V_1$ and $W_1$ are the base shear force and the total weight of the building, respectively.

Basic seismic index ($E_0$) is derived from expression (5). Final results are shown in Table 10. Irregularity index $S_D$ and Time index $T$ were taken as 1.0, therefore, the $E_0$ index equals the seismic capacity index $I_S$, according to expression (1).
4.6. Final results

Once every required index is calculated, it is possible to compare the seismic index of the structure ($I_S$) with the seismic demand index of the structure ($I_{S0}$) in order to complete the seismic vulnerability assessment by Index Method screening level 1. For a seismic action and structural behaviour derived from João Almeida’s pushover analysis of block #22, screening level 1 is not verified: $I_S$ index is exceeded by the $I_{S0}$ index in storeys 2, 3, 4, 6 and 7, as shown in Figure 15.

For a more straightforward comparison, Figure 16 shows final results by comparing, at each level, the shearing capacity ($V_{Rd,i}$) to the shearing forces due to seismic action ($V_{Sd,i}$). Obviously, seismic assessment results remain the same as shearing forces due to seismic action at five storeys.

Final results for shear strength ($V_{Rd,i}$) and shear stress ($V_{Sd,i}$) are shown in Table 11. Absolute shearing capacity usage is given by the difference of the previously mentioned indexes. The deficit of shearing capacity is higher at storeys 3, 4 and 6. For storeys 3 and 4, deficit of shearing capacity is due to the fact that a great percentage of mass is concentrated in those storeys, approximately 30% of total mass. The sudden building plan reduction in the 6th storey, which leads to an important decrease in shearing capacity at this level, is responsible for the deficit of shearing capacity.

### Table 11 – $V_{Rd,i}$ and $V_{Sd,i}$ value comparison for block #22 of SMH

<table>
<thead>
<tr>
<th>Storey</th>
<th>$V_{Sd}$ [kN]</th>
<th>$V_{Rd}$ [kN]</th>
<th>$V_{Rd} - V_{Sd}$ [kN]</th>
<th>(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>19396</td>
<td>21556</td>
<td>2160</td>
<td>10.0%</td>
</tr>
<tr>
<td>2</td>
<td>19220</td>
<td>19164</td>
<td>-56</td>
<td>-0.3%</td>
</tr>
<tr>
<td>3</td>
<td>18666</td>
<td>15901</td>
<td>-2765</td>
<td>-17.4%</td>
</tr>
<tr>
<td>4</td>
<td>17361</td>
<td>15655</td>
<td>-1706</td>
<td>-10.9%</td>
</tr>
<tr>
<td>5</td>
<td>15499</td>
<td>15631</td>
<td>131</td>
<td>0.8%</td>
</tr>
<tr>
<td>6</td>
<td>13112</td>
<td>10818</td>
<td>-2294</td>
<td>-21.2%</td>
</tr>
<tr>
<td>7</td>
<td>10697</td>
<td>10594</td>
<td>-103</td>
<td>-1.0%</td>
</tr>
<tr>
<td>8</td>
<td>7649</td>
<td>10285</td>
<td>2636</td>
<td>25.6%</td>
</tr>
<tr>
<td>9</td>
<td>3977</td>
<td>10130</td>
<td>6153</td>
<td>60.7%</td>
</tr>
</tbody>
</table>

4.7. $I_{S0}$ index derivation

An expression for Index $I_{S0}$ derivation was proposed on Section 3.3 Comparison with the results obtained in the pushover and detailed justification of the value assumed for each level follows, in order to assess its applicability.

$$I_{S0} = \frac{2.5 \cdot a_g R \cdot S \cdot \lambda_{X1} \cdot \gamma_1}{g \cdot q}$$

Peak ground acceleration ($a_g R$) is the defining parameter in the proposed expression. Response spectrum used in the pushover analysis was calibrated for a PGA value of 2.70ms$^{-2}$ and an elastic damping coefficient of 5%.

According the Output-Only Modal Testing and Identification [20], performed by Patricia Ferreira on block #22 of SMH, among others, the foundation soil is type B by EC8. The equivalent ground type, according to national code RSA [21], seems to be Type II soil. The soil parameter was taken as 1.0 because it was the value used on the pushover analysis for the response spectrum calibration.

Despite the fact that block #22 is part of a health facility, João Almeida opted to consider the importance factor ($\gamma_I$) value as 1.0 because the projected service life of the building (50 years) had already been achieved.

The behaviour factor ($q$) is related to the extent of the non-linear response of the structure According to section 5.2.2.2 of Part 1 of EC8, the minimum value of the behaviour factor is 1.5.

The value assumed for the correction factor ($\lambda$) is 1.0, as building does not fulfil the regularity requirements defined in section 4.2.3.3 of Part 1 of EC8, as follows:

$$0.15 \times H = 0.15 \times 32.15 = 4.82 \text{m} \Rightarrow \text{setback above 0.15H;}$$
Finally, value of $I_{S0}$ index is derived:

$$I_{S0} = \frac{2.5 \cdot a_{gr} \cdot S \cdot \lambda_{x1} \cdot \gamma_1}{g \cdot q} = \frac{2.5 \cdot 2.7 \cdot 1.0 \cdot 1.0}{9.81 \cdot 1.5} = 0.459$$

Under these circumstances, only storey levels 8 and 9 verify level 1 of the Index Method, as shown on Table 12. In this case, $\phi$ factor was computed according to the Index Method.

Table 12 – $V_{rd,i}$ and $V_{sd,i}$ value comparison for block #22 of SMH

<table>
<thead>
<tr>
<th>Storey</th>
<th>$\phi$</th>
<th>$C_c$</th>
<th>$C_{mar}$</th>
<th>$I_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>0.56</td>
<td>0.45</td>
<td>2.33</td>
<td>1.14</td>
</tr>
<tr>
<td>8</td>
<td>0.59</td>
<td>0.25</td>
<td>1.13</td>
<td>0.59</td>
</tr>
<tr>
<td>7</td>
<td>0.63</td>
<td>0.22</td>
<td>0.74</td>
<td>0.43</td>
</tr>
<tr>
<td>6</td>
<td>0.67</td>
<td>0.19</td>
<td>0.55</td>
<td>0.35</td>
</tr>
<tr>
<td>5</td>
<td>0.71</td>
<td>0.19</td>
<td>0.60</td>
<td>0.40</td>
</tr>
<tr>
<td>4</td>
<td>0.77</td>
<td>0.16</td>
<td>0.47</td>
<td>0.34</td>
</tr>
<tr>
<td>3</td>
<td>0.83</td>
<td>0.16</td>
<td>0.38</td>
<td>0.31</td>
</tr>
<tr>
<td>2</td>
<td>0.91</td>
<td>0.23</td>
<td>0.36</td>
<td>0.34</td>
</tr>
<tr>
<td>1</td>
<td>1.00</td>
<td>0.28</td>
<td>0.33</td>
<td>0.38</td>
</tr>
</tbody>
</table>

Figure 18 shows inertial forces, registered at each storey, by the pushover analysis and Index Method computation. Several similarities can be identified with the mass distribution graphic on Figure 20. Mass concentration between the 2nd and 4th storeys is responsible for greater inertial forces on these floors. Due to the setback on the 6th storey, where the building plan is reduced in 4.31m in the N-S direction, inertial force decrease is registered as a consequence of sudden mass reduction.

Shear force distribution along height given by the Index Method is a function of the shear modification factor ($\phi$) and mass distribution at each level. By comparison of the shear forces registered by the pushover analysis, the Index Method overestimates at lower storeys and underestimates at top storeys.

The Index Method assumes a regular inter-storey height which is definitely not the case of block #22. As an example, 1st storey is 2.90m high while the 3rd storey is 4.20 high, which is a 50% discrepancy. That difference has direct influence on the value of inertial forces, as they depend on the distance to ground level.
The proposed expression for $I_{S0}$ index derivation performed well by comparison with the pushover analysis.

Although the proposed expression lead to slightly higher seismic coefficient (0.459) by comparison of that registered by the pushover analysis (0.422) less demanding shear forces at upper storeys and more conservative shear forces on lower storeys were derived by the proposed expression. That fact is due to the $\phi$ factor assumption of a linear distribution of accelerations in height, in comparison with those registered in the pushover analysis.

![Figure 20 – Mass distribution per storey](image)

4.8. Assessment of behaviour coefficient on performance point

The behaviour factor ($q$) is related to the extent of the non-linear response of the structure; it depends on the assigned resistance and energy-dissipation capacity of the structure. The thorough assessment of the behaviour factor value proved to be a hard task. A brief explanation of the main steps towards its correct assessment follows.

Total damping ratio is the combination of the viscous damping ratio ($\xi$), taken as 5% for RC structures, and a damping ratio related to the energy dissipation capacity of the structure in hysteretic cycles, given it is exploiting its non-linear response. Response spectrum reduction, for structures with less than 5% of viscous damping ratio, is possible in EC8. The damping correction factor ($\eta$), which is part of elastic response spectrum defining expressions, has a reference value of 1 for 5% viscous damping structures, as shown on expression (24).

$$\eta = \frac{\sqrt{10(5 + \xi)}}{5 + \xi} \geq 0.55 \quad (24)$$

Initial approach to the behaviour factor value is made by first deriving the correction factor using viscous damping ratio of 13.88% that is the equivalent damping ratio obtained by the pushover analysis. Then, the behaviour factor equals the reciprocal of the damping correction factor ($\eta$), as shown on the following calculation.

$$\eta = \sqrt{10/(5+13.88)} = 0.7278 \quad \text{and} \quad \frac{1}{\eta} = 1.374$$

The base shear force value, obtained by the final expression of Derivation 2, was 14505.0 kN\(^6\), which is fairly high when compared with 7980.0 kN, that is the value of base shear coefficient obtained with the pushover analysis at the performance point.

While performing non-linear analysis, reduction of the inertial forces applied at each level and, therefore, the seismic action, occurs for two reasons. Firstly, the increase of the energy dissipation capacity of the structure, given it is exploiting its non-linear response (in hysteretic cycles), produces the same effect on the structure as a linear decrease of the response spectrum. This phenomenon can be simulated by using the damping correction factor ($\eta$) and is represented in Figure 15 by a shift from position “A” to “B”. Secondly, plastic hinges may form in the most stressed sections of structural elements; if rotation capacity of the plastic hinges is achieved, elements may collapse. If this phenomenon spreads to a considerable number of elements, it leads to a global stiffness loss, changes on modal shape and of the fundamental period. The direct consequence of period shift is to alter the corresponding spectral acceleration. This was the case of the pushover analysis, where the fundamental period was lengthened from an initial value of 0.488 s to 1.153 s, leading to a substantial decrease of the spectral acceleration. This phenomenon is represented, in Figure 21, by a shift from position “B” to “C”.

![Figure 21 – Elastic and reduced response spectra used for analysis.](image)

---

\(6\) Modal analysis was performed on block #22 using the SAP2000 model created by João Almeida. The effective modal mass registered at the 1st mode (fundamental) was 64%. Correction factor ($\lambda$) was taken as 0.64.
Position “A” refers to the spectral, acceleration registered for the fundamental mode, in the first stages of analysis, while still on linear response ($T_1=0.488 s$). Position “C” refers to the spectral acceleration, registered for the fundamental mode, at the performance point, over the reduced spectrum ($T_1=1.153 s$). The shift from position “A” to “C” is related to seismic action reduction effects previously described and occurs continuously while the analysis is being carried out. Therefore, using spectral reduction, by means of the correction factor ($\eta$), to simulate non-linear response of the structure in a pushover analysis, is an incomplete approach; it does not account for period shifting and the changing of the spectral acceleration value that occur as a consequence.

The behaviour factor is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic, to the seismic forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure. [12] In order to estimate accurately the behaviour factor, a linear dynamic analysis was performed, using the finite element model of block #22 created by João Almeida for the pushover analysis. The ratio of the base shear force derived in the linear dynamic analysis (20377.6 kN), to the base shear force obtained in the pushover analysis (7980.0 kN), is a measure of the non-linear response of the structure and equals the behaviour factor ($q = 2.655$).

5. Conclusions

Performing the seismic assessment of Block #22 of SMH with screening level 1 of the Index Method, taking into account the seismic action and modelling options adopted in João Almeida’s pushover analysis, demonstrated that some storeys are vulnerable, in the analyzed N-S direction.

For a comparison purpose, contribution of the infilled brick walls to shearing strength of each storey was derived by accounting the contribution of each strut beam used in the actual finite-element model. Overall shearing strength was computed according to Index method expressions.

The effects of floor plan irregularity, by suppression of columns on the 6th floor, and non-uniform floor mass distribution, as approximately 30% of building’s total mass is concentrated on only two floors (2nd and 3rd floors), are qualitatively represented by the behaviour of the $I_s$ index. Values of shearing capacity deficit are higher in the floors where the previously mentioned irregularities occur.

Pushover analysis results have shown that, although collapse is not imminent, there is a large possibility of extreme damage concentration between the 3rd and 4th storeys. At that level, the second highest deficit of shearing capacity was registered with 17%. Out of all the analyzed storeys, the highest value (21%) was registered at the 6th floor. Therefore, the Index Method was able to identify the 3rd and 6th floors as most vulnerable.

The proposed expression for $I_{oi}$ index computation was later tested. By comparing the shear stress forces per storey obtained by the Index Method with those given by the pushover analysis, it is possible to conclude that the conservativeness of Index Method does not lie on the shear modification factor ($\phi$), responsible for the shear stress forces distribution, but on the wise assumption of the values for the other coefficients, namely the behaviour factor ($q$). Major conclusions are similar to the assessment performed using shear stress obtained by the pushover analysis, except for the fact that the 7th storey only considered safe by the Index Method, only while using the proposed expression.

Although it lead to a higher value of the base shear force (0,459 against 0,422 of the pushover analysis), shear force distribution in height given by the shear modification factor $\phi$ proved conservative in the lower floors and the opposite in the upper floors.

Maximum over-strength registered was +60% and +61%, both at the 9th storey, according to the assessment performed using shear forces obtained from the pushover analysis and from the proposed expression, respectively. Also, the maximum strength deficit registered was -50% on the 6th floor (index method) and -21% on the 3rd floor (pushover).

The behaviour factor accounts for the energy dissipation capacity of the structure, being 1.5 the lowest possible according to Part 1 of EC8. The value of 2.56, obtained indirectly from the pushover analysis, shows greater energy dissipation capacity and is the result of two facts: on the one hand, the reduction of the response spectrum due to energy dissipation in the hysteretic cycles and, on the other hand, the reduction of spectral acceleration due to structure fundamental period increase by performing in non-linear regime.

It is within the scope of this work, among others, to present the Index Method, explain its basic assumptions, calculation procedure, to adapt it according to local seismic risk and common structural solutions. The adoption of basic concepts and values, present in Part 1 of EC8, in the $I_{so}$ index expression performed fairly well. The results achieved for the shear force distribution per storey by the proposed expression, when compared to those obtained by the pushover analysis, are quite similar.

The theoretical derivation performed for the shear modification factor ($\phi$) allowed to identify the basic assumptions behind its expression. The general expression presented by Index Method was obtained by considering a uniform floor weight distribution and inter-storey height constant.
obtained by the deducted expression for a factor assumes the value of 0.85. That was the value obtained by the deducted expression for a 3-storey building.

The great advantage of the Index Method lies in its simple and easy procedure. Assessing a structure according to Level 1 requires a minimum amount of information concerning the building (e.g., cross-sections of each structural element type, time-related deterioration, etc.) and can be performed with the simple use of a spreadsheet.

6. Suggestions for further research

Structural vulnerability performed by the Index Method is established by comparison of $I_s$ and $I_{S0}$ indexes. The success of the adaptation to Portugal depends on the correct assessment of both indexes.

Considering the expression proposed for $I_{S0}$ (seismic demanding index) effort must be made in order to estimate, as accurately as possible, the value of the behaviour factor, as it is vital for the correct assessment of the base shear force.

Several parameters take part in the calculation of $I_s$ (seismic performance index). Such parameters, as the average shear stress at ultimate limit state of each structural element ($\tau_i$) or the effective strength factor ($\alpha_i$), have a vital role on estimating the global shearing strength at each level. More precise assess of that strength can be achieved by parameter calibration, reflecting local materials used and standard detail designing, for common structural systems. The use of values proposed by PAHO [2], when compared to experimental results obtained by Felicita Pires [11] on concrete frame infilled with brick panels, showed the necessity for calibration, as the results were too conservative. A simple way to evaluate the accurateness of the values proposed by PAHO [2] is to compare them with the estimation of the shearing strength of each column of the SMH under compound bending. Calculation can be performed using the finite element model created for the pushover analysis.

Part 3 of EC8, entitled Assessment and Retrofitting of Buildings, addresses the seismic vulnerability assessment of buildings in seismic regions and specific rules for different materials as concrete, steel, and masonry. It would be interesting to compare the Index Method with the assessment method of reinforced concrete buildings in their present state, proposed in Annex A of Part 3 of EC8, regarding the following aspects: basic assumptions, ease of use and final results.

The use of the Index Method in Portugal on a large scale still depends on its application and calibration to buildings with different structural systems, as masonry buildings, RC framed buildings and RC dual frame-wall buildings. Calibration of the method for each structural system should be made by comparison with more precise analysis, like dynamic linear analysis or pushover analysis (static nonlinear) for example. The present work aimed to contribute to a data bank with an example of seismic vulnerability assessment using the Index Method applied to a RC framed structure infilled with non-confined brick walls.

The Index Method addresses three levels of assessment of the $I_s$ index that vary in calculation detail and amount of information required from the structure. Calculation’s precision depends on the level used. For instance, in level 2 of assessment by the Index Method, strength of structural elements (columns and walls) is calculated by shear failure and flexural yielding, while on level 1 only shear failure is considered. Also, the ductility level of those failure modes is roughly estimated, based on the basic principles of capacity design. Even more detailed calculation is used in level 3 of assessment. Early failure of beams (shear or flexural yielding), connecting vertical elements, is considered. RC Walls contribution to global strength is considered as continuous, from foundation to top, instead of separately for each level.

References


