Application of the ICIST/ACSS methodology variant for Masonry Buildings with RC Slabs

Case Study: Central Pavilion of the Instituto Superior Técnico

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Abstract

Seismic activity in Portugal has been a recurring subject in recent times, with the prospect of occurring again an earthquake like the one that devastated a large portion of downtown Lisbon in 1755, although an earthquake of this type has a large return period. Due to the large concentration of old buildings in Lisbon, it becomes necessary to know which buildings potentially require interventions or should be demolished. Some of these vulnerable buildings, however, are relatively modern, due to the first Portuguese seismic regulation having been implemented only in 1958. The assessment of the seismic vulnerability of these buildings can be time-consuming and costly, involving the development of refined numerical models, stressing the need for expeditious methodologies to evaluate the seismic vulnerability of a building. The ICIST/ ACSS methodology consists of a simple but sufficiently rigorous method that allows the comparison of a seismic performance index and a seismic demand index of the structure. This methodology is based on the Hirosawa method developed by the Ministry of Construction of Japan, and includes three levels of evaluation of increasing complexity, with the ICIST/ACSS methodology based on the first level. In this dissertation, a variant of the ICIST/ACSS methodology is taken up, initially developed by Filipa Chaves and Jorge Proença for use in masonry buildings with reinforced concrete slabs or "*Placa*" buildings. The case study is the Central Pavilion of the Instituto Superior Técnico, an iconic and representative building of the *Estado Novo* Architecture in Portugal.

Keywords: methodology, seismic vulnerability, Central Pavilion, masonry

1. Introduction

The city of Lisbon has a large concentration of old buildings, especially in areas close to the city's coast. Although it adds value for the conservation of the Portuguese historical heritage, it is worrying that the great majority of the old Portuguese buildings are non-compliant to current Earthquake design standards, and could potentially collapse in the face of a high intensity earthquake.

Prior to the widespread adoption of reinforced concrete in structures in Portugal in the mid-twentieth century, there were three predominant types of buildings. These are the *pombalino* buildings, *gaioleiro* buildings and *placa* buildings, consisting mostly of masonry walls and timber floors and joists, except for the *placa* buildings which contain RC slabs.

The scope of this paper is to adapt the ICIST/ACSS methodology to *placa* buildings. The ICIST/ACSS methodology [1] was developed by the Institute of Structural, Territorial and Construction Engineering of Instituto Superior Técnico (ICIST), as a request from the Central Administration of the Ministry of Health (ACSS), and consists of a method to assess the seismic vulnerability of reinforced concrete structures. This method is based on the first level of assessment of the Hirosawa method [2] developed by the Ministry of Construction of Japan. The adaptation of the ICIST/ACSS method has been previously undertaken by Filipa Chaves and Jorge Proença [3], however this adaptation is an ongoing work in process which ultimately aims to adapt this methodology to a multitude of structure types. The case study for this paper is the Central Pavilion of the Instituto Superior Técnico, which consists of a *placa* building due to its masonry walls reinforced concrete slabs and some reinforced concrete columns and beams. One of the main assumptions of the ICIST/ACSS method, which is to consider a rigid diaphragm behaviour in each floor, is still valid for *placa* buildings due to the presence of slabs in reinforced concrete.

2. Review of Seismic Behaviour of masonry buildings with RC slabs

The structural behaviour of a wall can be modelled as a deep beam. The formula for the overall stiffness (bending and shear) of a wall is given by equation (1).

$$K_{total} = \frac{E.b}{\left[4\left(\frac{H}{L}\right)^3 + 3\left(\frac{H}{L}\right)\right]} \quad (1)$$

where:

- E is the Young's modulus of the material of the wall;
- b is the thickness of the wall;
- H is the height of the wall;
- L is the length of the wall.

In equation (1), the component $4\left(\frac{H}{L}\right)^3$ reflects the bending stiffness influence, whereas the component $3\left(\frac{H}{L}\right)$ is associated with the shear stiffness. The presence of openings in the wall significantly influences its stiffness. Its influence has been studied by Neuenhofer [4]. In his study, a comparison is made between the stiffness of walls with openings in several arrangements and dimensions obtained by the finite element method, with those obtained via a simplified manual method.

Neuenhofer's findings show that the presence of an opening in a wall may reduce its stiffness by up to 50% when compared to its stiffness without openings, whereas by the simplified manual method the rigidity obtained is significantly higher.

The mechanical properties of masonry depend mainly on its type of construction, state of conservation, the type of mortar and the characteristics of stone, masonry or brick. However, despite the variety of values, masonry is generally associated with poor tensile strength, reasonable shear strength when combined with compression and high compressive strength, which can be represented by Coulomb's law – equation (2).

$$\tau_{Rd} = c_u + \sigma_0 \tan\phi' \quad (2)$$

It is, however, recommended that the mechanical parameters of masonry should be obtained through in-situ experimental tests, which may be destructive or non-destructive. If it is not possible to carry out on-site tests, these parameters must be obtained by consulting the literature, namely technical tables and standards, contemporaneous with the date of construction of the building being studied. Such standards include, for example, the New Italian Technical Norm [5] and the *Normativa* Italian standard [6], whose indicated values for masonry have been considered for the case study of this paper.

When it comes to failure modes of masonry walls, two main groups of failure modes are considered: out-of-plane and in-plane of the wall. Failure modes outside the plane of the wall occur when the walls are not properly connected between them or with annexed floors. There are four types of failure mode outside the plane of the wall: rocking, articulated rocking, vertical bending and horizontal bending [7].

When it comes to in-plane failure mechanisms, there are four different types of failure: sliding, diagonal cracking and rocking, which can be accompanied by toe-crushing. These failure mechanisms, unlike out-of-plane failure modes, occur when the walls are properly connected between them, and the likelihood of each of these occurring depend on factors such as the wall slenderness, boundary conditions and axial compression on the wall.

The equations (3), (4) and (5) enable to calculate the resisting shear stress of failure modes by sliding, diagonal cracking and rocking or toe-crushing, respectively.

$$\tau_{Rd} = \frac{1.5c_u + \sigma_0 \tan\phi'}{1 + \frac{3h_0 . c_u}{\sigma_0 . b}} \quad (3)$$

$$\tau_{Rd} = \frac{1.5c_u}{\beta}\sqrt{1 + \frac{\sigma_0}{1.5c_u}} \quad (4)$$

$$\tau_{Rd} = \frac{\sigma_0 . b}{2h_0}\left(1 - \frac{\sigma_0}{k . f_d}\right) \quad (5)$$

where:

- c_u is the material cohesion;
- σ_0 is the compressive stress on the wall;
- ϕ' is the internal friction angle of the material;
- b is the length of the wall;
- h_0 is the distance between the section of the null moment and the base of the wall (which is considered equal to the distance between the base and the top of the wall for walls situated on the last storey level and half of that number for walls situated on middle storey levels);
- β is the relation between the distance between the top section and the base section of the wall (h) and the length (b). This parameter is inferiorly limited by 1.0 and superiorly by 1.5;
- k is the factor that turns the linear distribution of the compressive stress into a rectangular diagram (assumed to be equal to 0.85);

3. Seismic vulnerability assessment methods

There is currently an extensive array of available seismic vulnerability assessment methodologies, each requiring different amounts of information, detail and complexity.

On the one hand, there are generalized methodologies, such as the HAZUS methodology developed by the Federal Emergency Management Agency [8] of the Government of the United States of America, or methodologies based on the European Macro-Seismic Scale (EMS 98) [1]. The assessment in these methodologies is done in a qualitative fashion and are appropriate for risk assessment at the urban level, however, they can be misleading due to the fact that they are not based on quantitative data, such as structural element dimensions. On the other hand, there are more detailed methodologies, based on the design and seismic design according to Eurocode 8, naturally implying a high expenditure of resources, although slightly less than the resources associated with a possible reinforcement intervention. An evaluation of this type is therefore contradictory if the aim is to avoid costs associated with an intervention.

There is, therefore, a need to incorporate an intermediate scale methodology which requires a significant amount of information from the structure but is relatively expeditious. Seismic assessment methodologies such as the Hirosawa method and the *Vulnerabilità Muratura* [9] method match this description. The methodologies covered in this chapter consist of variants of the Hirosawa method, these are the ICIST/ACSS Method [1], the variant proposed by the PAHO - Pan American Health Organization [10] and the ICIST/ACSS variant proposed for masonry buildings with RC slabs, whose development was made by Filipa Chaves in her dissertation [3].

The ICIST/ACSS methodology [1] results of the adaptation of the Hirosawa method for the Portuguese context, developed for reinforced concrete buildings. The assessment is undertaken by comparing the indexes of seismic performance of the structure (I_S) and the index of the seismic solicitation of the structure (I_{S0}). Seismic safety of the structure is verified when $I_S \geq 1.2\, I_{S0}$. If $I_S < 0.8\, I_{S0}$, the building may show an unsatisfactory structural behaviour facing an earthquake, being considered unsafe. There is a third scenario, in which $0{,}8.I_{S0} \leq I_{Sn} < 1{,}2.I_{S0}$. In this case, it is considered that the analysis is inconclusive and it is recommended that further studies are undertaken.

The index of the seismic performance of the structure (I_S) is determined to each storey level and each main horizontal direction of the building through the following equation:

$$I_S = E_0.S_D.T \tag{6}$$

where:

- E_0 is the sub-index of the seismic performance of the structure;
- S_D is the sub-index of structure irregularity;
- T is the sub-index of structural deterioration.

The index of the seismic solicitation of the structure (I_{S0}) is calculated for the whole building through the following equation:

$$I_{S0} = \frac{S_d(T_1).\lambda_1.\chi}{g} \tag{7}$$

in which:

T_1 is the fundamental period of the structure to translation movements in a given horizontal direction;

$S_d(T_1)$ is the spectral acceleration, calculated according to National Annex of Eurocode 8;

λ_1 is the percentage of mobilized mass in the fundamental mode of vibration under analysis in a given horizontal direction;

χ is a reduction coefficient to apply in case the considered design life is different than 50 years;

g is the acceleration of gravity which is 9.8 m/s².

The sub-index of the seismic performance of the structure E_0 considers the resistant elements of the building, which are classified as columns (C), short columns (SC) and walls (W). Each of these is further sub-classified into different elements, depending on their slenderness and section dimensions ratio, each of them with an associated performance sub-index. The methodology also considers structural masonry walls (M) and reinforced structural masonry walls (MR).

The calculation for each of these structural elements' performance sub-indexes is given by the generalized formula – equation (8).

$$C_i = \frac{\sum A_i \tau_i}{\sum W}\beta_C \tag{8}$$

where:

- A_i is the section area of the element i;
- τ_i is the average resistant shear stress of the element i;
- $\sum W$ is the sum of the weight of the building floors above a given floor;
- β_C is a correction factor applied to the concrete elements should the characteristic compressive stress f_c be different than 20 MPa, and is given by $f_c/20$ if $f_c > 20\ MPa$ and $\sqrt{f_c/20}$ if $f_c < 20\ MPa$.

The methodology proposed by the PAHO, like the ICIST/ACSS method, is based on the first level of assessment of the Hirosawa method [10], with the inclusion of performance sub-indices for types of masonry walls not contemplated in the original methodology. These additional wall types considered are infilled brick walls (MAR), unreinforced or partially confined brick walls (A) and confined brick walls (MA), the latter of which includes the influence of the vertical load present on the masonry wall in its formula – equation (9).

$$C_{ma} = \frac{0{,}6.(0{,}45.\tau_0 + 0{,}25.\sigma_0).A_{ma}}{W} \times 1000 \tag{9}$$

where:

- A_{ma} is the sum of the section areas of the infilled brick walls in a given floor;
- τ_0 is the masonry cohesion value;
- σ_0 is the axial stress present in the infilled brick walls due to the vertical loads;
- W is the total weight of the building above a given floor.

Finally, there is the proposed variant of the ICIST/ACSS methodology for masonry buildings with RC slabs. This methodology, developed by Filipa Chaves and Jorge Proença [3], has a slightly different approach when compared to the ICIST/ACSS method, in the sense that instead of comparing indexes, the comparison is between resisting and acting shear forces (F_{Rd} and F_{Sd}). This methodology allows for four different types of seismic vulnerability assessment: global, by alignment, by wall panel and by wall element. Wall panel defines a continuous resistant element of constant thickness and wall element has a length equal to the distance between openings and a height equal to the distance between the base and the top of the wall. An alignment corresponds to a group of resistant elements that are located in the same vertical plan through a horizontal direction of the considered storey level. The proposed methodology was developed for buildings with rigid diaphragms and efficient connections between orthogonal walls and between walls and floors, considering only the in-plane collapse mechanisms.

The resisting shear force at the level of a floor is calculated in each main horizontal direction of the building and results from the sum of the resistive forces of the wall elements located in the horizontal direction under analysis – equation (10).

$$F_{Rd} = \sum_{i}^{n} \alpha_i . A_i . \tau_i \tag{10}$$

where:

- n is the number of wall elements of the direction of the storey level under analysis;
- α_i is a reduction factor of the resistant capacity of the wall element i which varies according to the predictable failure mode of the structure. A unit value for all structural masonry walls is considered, no distinction being considered between the different modes of collapse of a masonry wall. Its study is not within the scope of this paper, but it is recommended to calibrate this factor in later studies;

A_i – is the transversal section area of the wall element;

τ_i – is the ultimate resisting shear stress of the wall element.

The ultimate resisting shear stress of the wall element assumes the lowest value of the ones obtained for each of the three considered collapse mechanisms: sliding shear failure, diagonal cracking and rocking or toe-crushing. This value corresponds to the collapse mechanism that anticipates the rupture of the wall element. The equations that enable to calculate the resisting shear stress for each of the considered collapse mechanisms are, respectively, equations (2.1), (2.2) and (2.3) presented in 2.

The acting force F_{Sd} is calculated in a manner which resembles the index of seismic solicitation of the structure, and it is calculated for each storey level of the building according to the equation (4.2), assuming the same value in the two main horizontal directions.

$$F_{Sd} = \frac{S_d(T_1) . \lambda_1 . \chi}{g} . \frac{W}{\phi . S_D . T} \tag{11}$$

in which:

- W is the total weight of the building above the storey level in analysis;
- ϕ is the modification factor given by $\frac{n+1}{n+i}$ where n corresponds to the total number of storey levels of the building under evaluation and i corresponds to the storey level under analysis;
- S_D is the sub-index of structural irregularity;
- T is the sub-index of temporal deterioration (which assumes a value of 0.7 for buildings with displacements on the foundations, 0.9 for buildings that present cracks on the walls and 1.0 for buildings without signs of deterioration);
- λ_1 is the percentage of mobilized mass in the fundamental mode of vibration under analysis in a given horizontal direction;

The Eurocode 8 recommends a percentage of mobilized mass λ_1 of 0.85 for a building with more than two floors and with a fundamental period of the structure less than or equal to two times the period corresponding to the superior limit of the constant acceleration portion of the design response spectrum, or T_C.

In order to perform an analysis by alignment, wall panel or wall element, it is required to distribute the seismic force along each of these elements that are aligned in the horizontal direction under analysis. For this purpose, three methodologies for seismic force distribution are proposed: by cross-sectional area, by shear stiffness, and by global stiffness - equations (12) (13) and (14).

$$F_{Sd_i} = A_i \frac{F_{Sd}}{\sum A_i} \tag{12}$$

$$F_{Sd_i} = K_{shear_i} \frac{F_{Sd}}{\sum K_{shear_i}} \tag{13}$$

$$F_{Sd_i} = K_{global_i} \frac{F_{Sd}}{\sum K_{global_i}} \tag{14}$$

in which:

- F_{Sd} is the acting force in the storey level being studied;
- F_{Sd_i} is the acting force in the wall element i;
- A_i is the cross section area of the wall element i;
- K_{shear_i} is the shear stiffness of the wall element i;
- K_{global_i} is the global (shear and bending) stiffness of the wall element i.

The stiffness equations can be obtained as mentioned in 2.

4. Case Study: The Central Pavilion of the Instituto Superior Técnico

The development of the Instituto Superior Técnico project took place between 1927 and 1935 by renowned Portuguese architect Porfirio Pardal Monteiro, as a request from the also renowned Portuguese engineer Duarte Pacheco [11]. Its construction was concluded in 1941 and it is considered a prime example of the *Estado Novo* architectural movement entitled *Português Suave*.

Figure 1: Aerial photograph of the Instituto Superior Técnico circa 1934

The construction system used in the Central Pavilion, as in the other buildings, is a mixed system of masonry walls made of brick or stone with thin slabs and the occasional columns and beams in reinforced concrete [12]. It is, for all intents and purposes, a *placa* building, as it is composed of a concrete slab that discharges into masonry walls. It consists of three main floors: the basement, the ground floor and the first floor. Due to the considerable height between floors of the North and South wings (5m), additional intermediate floors have been introduced in some sections of the Pavilion, on the ground floor and on the first floor, through mezzanines composed of a metallic structure. These intermediate floors contain several offices and academic nuclei.

A numerical model has been produced for the Central Pavilion of the Instituto Superior Técnico using the structural analysis software SAP2000, based on architectural blueprints and information gathered from architectural and engineering journals and articles made available through historical archives located in the Instituto Superior Técnico, namely the NArQ (*Núcleo de Arquivo do IST*). Ideally, this model would be based on the structural drawings produced by the appointed structural engineer at the time of its construction, however, these could not be retrieved in any of the visited historical archives throughout Lisbon. Therefore, a number of assumptions had to be made in order to produce a relatively robust numerical model which is representative of its behaviour towards earthquakes.

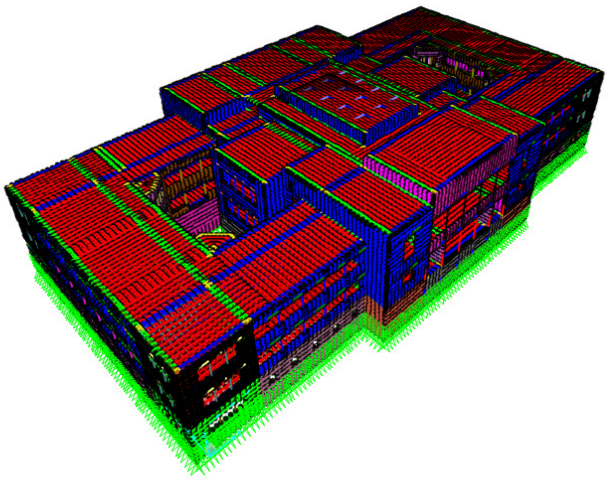

Figure 2: Three-dimensional finite element model of the Central Pavilion of the Instituto Superior Técnico developed in SAP2000

The maximum dimension of the shell elements is 0.8x0.8m. To correctly model the interface behaviour between reinforced concrete slabs and masonry walls, a stiffness modifier was applied to the bending moment m_{11} (or m_{11} depending on the direction) near zero in the shells of the slabs where they meet the masonry walls. This way, no bending moment is transmitted from the slab to the wall about the horizontal axis in the plane of the wall. These sections are shown in blue, green and yellow on the slabs in Figure 2.

The modelling of the interaction of the building with the ground is done through the application of conditions of support, or restraints, in the buried masonry walls. For this purpose, the following hypotheses were considered:

- The buried walls are only allowed to exhibit in-plane displacements, therefore translation restrictions were applied in the nodes of their shell elements in the direction perpendicular to the plane of the wall;
- Only the walls of the periphery of the basement of the Central Pavilion and the atrium are considered as buried walls;
- The interior walls of the basement have translation and rotation restrictions at its base;
- For the purpose of simplifying the model, the ground level of the building is considered as the ground level of its lateral entrances

The values of the physical and mechanical parameters adopted for the structural materials - Table 1 and Table 2 - consist of average values for stone masonry, available in the New Italian Technical Norm [5]. The choice of average values for the numerical model is justified by the fact that the available literature reveals the presence of stone or brick masonry, and it is not possible to accurately identify the type of masonry present without using in situ tests. Given the time of construction of the Instituto Superior Técnico, at a time when constructions in reinforced concrete were not the norm, the class of reinforced concrete considered is equivalent to the current class C16/20 – Tables 1 and 2.

Table 1: Self weight and Young's modulus of the structural materials

Material	E (GPa)	w (kN/m^3)
Stone masonry	2	23
Reinforced concrete C16/20	29	25
Structural steel (skylight)	200	77

Table 2: Mechanical properties of the stone masonry

Parameters	Value
Cohesion (c_u)	0,06 MPa
Resistance to compression (f_d)	1,8 MPa
Coefficient of friction (tan ϕ')	0,4

For the definition of the other permanent loads, the building materials, such as cladding of all the structural elements and the aforementioned mezzanines, have been considered in the model, considering values suggested in *Tabelas Técnicas* [13]. The variable loads have been defined as per the Eurocode 1 Part 1-1, and vary according to the different uses of the Central Pavilion rooms and sections – Figure 3. The partition walls were also only considered as other permanent loads. The criterion for consideration of the structural modelling of a wall in the numerical model was given by its thickness and the continuity of the wall along the upper floors, since these walls transfer forces to the lower floors. Therefore, walls with thicknesses greater than 0.25 m with continuity to the upper floors were considered as structural walls.

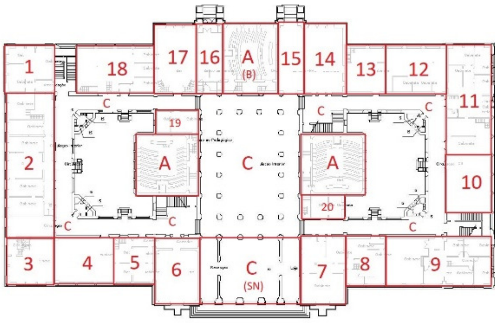

Figure 3: Zoning considered for the permanent and variable loads in the Central Pavilion

The quantification of the seismic action is done according to Eurocode 8 and its National Annex. The seismic zone is considered to be Lisbon, the type of foundation ground is B, the importance class is III, and the behaviour coefficient is 1.5. Table 3 summarizes the parameters considered for the quantification of the seismic action.

Table 3: Parameters for the definition of the design response spectrum of the Central Pavilion in accordance with Eurocode 8

Parameter	Type 1 Earthquake	Type 2 Earthquake
Seismic zone (Lisbon)	1.3	2.3
$\mathbf{a_{gR}}$ **(m/s²)**	1,5	1,7
Terrain type	B	
$\mathbf{S_{máx}}$	1,35	1,35
$\mathbf{T_B}$ **(s)**	0,1	0,1
$\mathbf{T_C}$ **(s)**	0,6	0,25
$\mathbf{T_D}$ **(s)**	2,0	2,0
γ_I	1,45	1,25
$\mathbf{a_G}$ **(m/s²)**	2,175	2,125
S	1,213	1,219
q	1,5	

The combination of actions considered is in accordance with Eurocode 8 – Equation (15).

$$\sum G_{k,j} + \sum \psi_{E,i}.Q_{k,i} \tag{15}$$

where:

- $G_{k,j}$ is the characteristic value of the permanent action j;
- $\psi_{E,i}$ is the combination coefficient of the variable action i for seismic actions, and is equal to $\varphi.\psi_{2i}$ where φ is a parameter which converts the quasi-permanent combination coefficient into a combination coefficient for seismic situation projects;
- $Q_{k,i}$ is the characteristic value of the variable action j.

Table 4 summarises the considered actions and respective combination coefficients.

Table 4: Summary of actions and coefficients considered in the Central Pavilion numerical model

Action		φ	ψ_{2i}	$\psi_{E,i}$
G_k	**Self weight**	-	-	-
	Other permanent loads	-	-	-
Q_k	**Surcharge: Category B**	0,8	0,3	0,24
	Surcharge: Category C	0,8	0,6	0,48
	Surcharge: Roof	1,0	0	0

The undertaken dynamical analysis is one of response spectrum analysis, which takes into account the several modes of vibration which contribute significantly to the global response of the structure in analysis. Table 5 summarises the values for the period and participation factors of the main modes of vibration of the structure, obtained from the SAP2000 output of the numerical model.

Table 5: Vibration modes and respective modal participation factors for translation in the x and y directions and rotation about the z axis

Mode	Period (s)	UX (%)	UY (%)	RZ (%)	∑ UX (%)	∑ UY (%)	∑ RZ (%)
1	0,24	0	**60**	4	0	60	6
2	0,23	**62**	0	1	63	60	6
3	0,22	1	5	**57**	64	65	63
4	0,17	0	**2**	0	64	67	63
…	…	…	…	…	…	…	…
50	0,10	0	0	0	67	72	66

Looking at the values in Table 3, it is possible to ascertain that mode 1 corresponds to the first mode of translation in the y axis, mode 2 is the first mode of translation in the x axis, mode 3 is the first mode of rotation about the z axis and mode 4 is the second mode of translation in the x axis.

In an attempt to validate the values obtained in Table 5, an ambient vibration survey has been undertaken with an accelerometer in different locations along the roof of the Central Pavilion. The obtained values show that the likely natural frequencies of the building fall in the interval between [4.5Hz;5.8Hz], and the fundamental frequency obtained through the numerical model is approximately 1/0.24= 4.16Hz, with the following four vibration modes having greater frequencies up to 1/0.17=5.88Hz. Therefore, these values suggest an accurate numerical model.

The output obtained for the horizontal forces for the horizontal reactions of the building are shown in Table 6. Due to the main modes of vibration of the building in both directions falling within the plateau of constant acceleration of the design response spectrum, which means the Type 2 earthquake is the conditioning earthquake.

Table 6: Horizontal base reactions in the numerical model for a type 1 Earthquake

Seismic action	Global FX (kN)	Global FY (kN)
Type 1 Earthquake Direction x	**115481**	7383
Type 1 Earthquake Direction y	7414	**113916**

A total of three floors have been considered for the analysis, defined by z = -3.1m, z = 0.9m and z = 5.8m, where z denotes level – Figure 4.

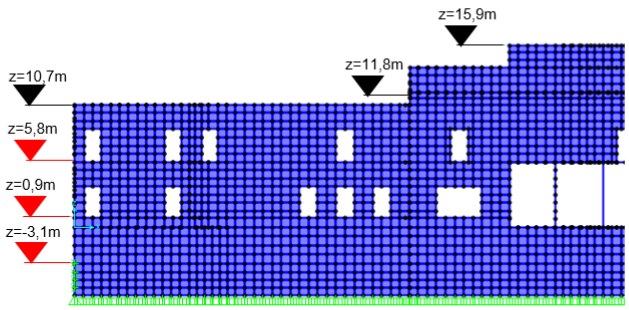

Figure 4: Levels considered for the analysis of the Central Pavilion of Instituto Superior Técnico, shown in red

The resistance of the building to the seismic action in each of the floors is given by equation (16).

$$F_{Rd} = \sum_{i=1}^{n} \alpha_i . A_i . \tau_i . F \tag{16}$$

This equation, when compared to equation (10), introduces the factor F which is a ductility index associated with the failure mode when calculating the sub-index E_0 in equation (6). The value for this index, along with the values for α_i are given in Table 7.

Table 7: Reduction factors and ductility index for the different failure modes

Type	α_1	α_2	α_3	F	Failure mode
A	1	0,7	0,5	0,8	Failure conditioned by short columns or structural masonry walls
B	0	1	0,7	1	Failure conditioned by reinforced concrete walls
C	0	0	1	1	Failure conditioned by reinforced concrete columns

The structural elements considered for the calculation of the resisting force F_{Rd} are summarised in Table 8.

Table 8: Structural elements of the Central Pavilion considered for the calculation of the resisting force F_{Rd} in accordance with the ICIST/ACSS methodology variant

Structural element	α_i	τ_i (MPa)	F
Masonry Walls - MA	1,0	$0,6.(0,45.\tau_0 + 0,25.\sigma_0)$	0,8
Reinforced Concrete Columns - C1	0,5	$\tau_c.\beta_c = 1.0\times0,77 = 0,77$	

A value of 1.0 MPa is recommended in the Hirosawa method for the resisting stress of C1 concrete columns. The τ_0 is as per Table 2. The value for σ_0 is directly obtained from the SAP2000 output, taking into account that the vertical forces in the reinforced concrete columns must not be considered, only the vertical force absorbed by the masonry walls. These values are affected by a safety factor of 1.35 which according with the New Italian Technical Norm [7] corresponds to a level of acquired knowledge $LC1$ (limited on-site surveys).

The results for the global analysis of the Central Pavilion are shown in Table 9.

Table 9: Global analysis of the Central Pavilion based on the results from the numerical model

Direction	Floor	F_{Sd} (kN)	σ_0 (kN/m²)	F_{Rd} (kN)	SF
x	z=-3.1m	115481	296.26	43237	**0.37**
	z=0.9m	102894	245.35	28399	**0.28**
	z=5.8m	67252	133.22	23905	**0.36**
y	z=-3.1m	113916	296.26	38739	**0.34**
	z=0.9m	99874	245.35	27711	**0.28**
	z=5.8m	65696	133.22	23220	**0.35**

where SF stands for safety factor.

The findings shown in Table 9 reveal a lack of overall safety of the Central Pavilion, in both directions and in all floors, to the seismic action defined in Table 3. It is also revealed that the safety factors factors are very similar in the two main horizontal directions. This is mainly due to the assumption that the axial tension in the masonry walls is equal in both directions. It is also due to the fact that the spectral accelerations in both directions are in the constant spectral acceleration plateau, resulting in equal $S_d(T_1)$ values in both directions and, therefore, in quite similar seismic forces.

In order to undertake an analysis by alignment, there is a need to define alignments along the different floors of the Central Pavilion. Figure 5 shows the alignments defined for this paper.

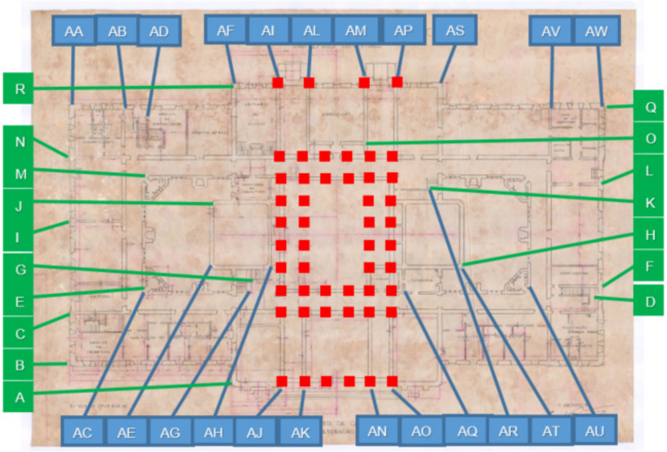

Figure 5: Alignments and columns considered for the analysis of the Central Pavilion (Columns shown in red)

The alignments indicated in Figure 5 are the alignments considered for the level z = -3.1m, not all of which have continuity for the upper floors. The total number of columns of the building is 50 and they all have continuity to the upper floors.

The results obtained in the analysis by alignment confirm that only the alignments AK and AN of the floor z = 0.9m verify the safety to the seismic action, while alignments AK and AN of floor z=-3.1m and the alignments A and R of floor z=5.8m show inconclusive results. The remaining alignments all have considerably reduced seismic safety factors. Truthfully, AK and AN alignments consist of only one masonry wall and 10 abutments. For this reason, on the one hand the absence of walls causes the stiffness to cut of this alignment to be inferior with respect to the other alignments in that direction, attracting less shear force. On the other hand, the resistance to the seismic action in this alignment relies mostly on the reinforced concrete columns, which compared to the resistance conferred by the masonry walls is much higher. An example of the full results obtained for the analysis by alignments of a floor will be shown in Chapter 5 when comparing them with the results obtained by the ICIST/ACSS methodology variant.

5. Case Study: Adaptation of the ICIST/ACSS methodology variant

This chapter adapts the variant of the ICIST/ACSS methodology proposed by Filipa Chaves, based on the behaviours and methodology proposed by PAHO. The analysis levels adopted for this analysis are at the global level and by alignments, considering the alignments defined in 4. Subsequently a comparison is made between the values obtained by each of the levels of the analysis by the variant of the ICIST/ACSS methodology and the values obtained through the numerical model. In order to apply the ICIST/ACSS variant to the Central Pavilion, it is necessary to consider a number of adaptations due to the presence of reinforced concrete columns in the different alignments, in particular regarding the distribution of vertical force by the different structural elements.

Table 10 shows the acting force F_{Sd} obtained via the ICIST/ACSS methodology variant, in accordance with equation (11).

Table 10: Acting forces per floor in the main horizontal directions according to the ICIST/ACSS methodology variant

Dir.	Floor	$S_d(T_1)$ (m/s²)	λ_1	ϕ	$F_{Sd}{}^{IA}$ (kN)
x	z=-3.1m	4.89	0.85	1.0	139370
	z=0.9m			0.8	113777
	z=5.8m			0.67	74482
y	z=-3.1m			1.0	139370
	z=0.9m			0.8	113777
	z=5.8m			0.67	74482

where $F_{Sd}{}^{IA}$ denotes acting force in accordance with the ICIST/ACSS methodology variant.

As it is intended to compare the results of the assessment through the ICIST/ACSS methodology variant with the results obtained in the dynamic analysis of the finite element model, the design life factor χ, the sub-index of temporal deterioration T and the sub-index of structure irregularity S_D have been assigned a unit value 1.0, since the numerical model does not consider these factors in its analysis.

The security of the building is then checked according to the variant of the ICIST/ACSS methodology. For this purpose, it was necessary to consider a method to distribute the vertical loads, for the combination of seismic design situations of Eurocode 0, by the masonry walls. It is therefore necessary to consider the influence areas of the columns in order to remove the vertical forces absorbed by the columns to the total weight above this floor. Approximately, the influence area of the columns of the Central Pavilion considered, for purposes of global analysis, is described in Figure 6.

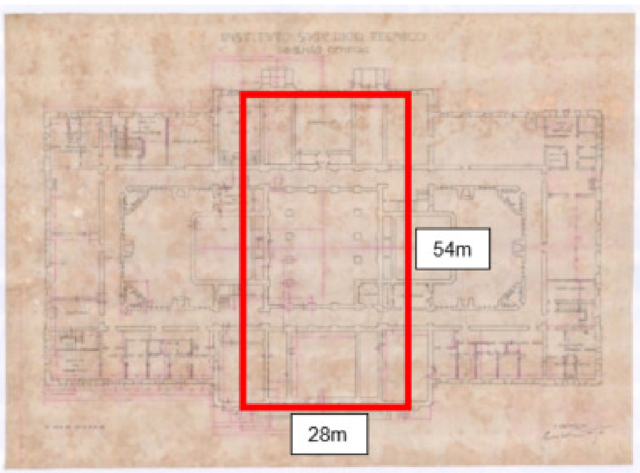

Figure 6: Alignments and columns considered for the analysis of the Central Pavilion (Columns shown in red)

Once the columns influence area is defined, the distribution of the vertical forces to the columns is made according to their axial stiffness. This value is then removed to the total vertical load applied in the floor being analysed, in order to obtain the axial stress σ_0 in the masonry walls to allow for the calculation of F_{Rd}.

Table 11 summarises the values obtained for the global analysis of the Central Pavilion in accordance with the ICIST/ACSS methodology variant.

Table 11: Global analysis of the Central Pavilion based on the ICIST/ACSS methodology variant

Direction	Floor	${F_{Sd}}^{IA}$ (kN)	σ_0 (kN/m^2)	${F_{Rd}}^{IA}$ (kN)	SF
x	z=-3.1m	139370	319.65	44447	**0.32**
	z=0.9m	113777	302.49	30333	**0.27**
	z=5.8m	74482	174.39	25249	**0.34**
y	z=-3.1m	139370	319.65	39669	**0.28**
	z=0.9m	113777	302.49	29523	**0.26**
	z=5.8m	74482	174.39	24431	**0.33**

The results obtained in Table 11 show that the variant of the ICIST/ACSS methodology is slightly conservative at a global level. This is mainly due to the percentage mass factor mobilized being equal to 0.85, rather than the actual mobilised mass obtained in the horizontal direction being studied from the numerical model, which increases the values of ${F_{Sd}}^{IA}$. Although the formula for obtaining the shear strength of the building is the same in the building analysis by the numerical model and by the ICIST/ACSS methodology, the values of F_{Rd} are slightly different due to the different ways of obtaining the vertical forces in the masonry walls. However, the results obtained in both approaches are considerably similar.

For the alignment analysis by the ICIST/ACSS methodology variant, it is necessary to distribute the seismic load forces of the global analysis from the ICIST/ACSS methodology variant ${F_{Sd}}^{IA}$ by the different alignments, as well as the vertical forces in each alignment due to the weight of the building above the floor being analysed.

The distribution of the seismic force by the alignments is made considering equation (13) for distribution of forces through the shear stiffness of the walls.

$$F_{Sd_i} = K_{shear_i} \frac{F_{Sd}}{\sum K_{shear_i}} \tag{13}$$

For the calculation of the shear stiffness of an alignment, the columns in the alignment must be considered. Therefore, the total stiffness of the alignment is given by the sum of the wall shear stiffnesses and the column shear stiffnesses. The shear stiffness of a wall is obtained by removing the bending stiffness component of the wall in equation (1), and a stiffness reduction factor is also applied which takes into account the presence of openings in the wall. This results in equation (17).

$$K^{w}_{shear,i} = \frac{E_a . b}{3\left(\frac{H}{L}\right)} . f \tag{17}$$

where:

- $K^{w}_{shear,i}$ is the shear stiffness of the wall;
- E_a is the Young's modulus considered for the stone masonry (2.0 GPa – Table 1);
- b is the wall thickness;
- H is the wall height;
- L is the wall length;
- f is a factor which takes into account the presence of openings in the wall, considered equal to 1.0 for walls without significant openings. For the present paper, a value of 0.5 is considered if a wall has openings greater than 1/3 of the total wall area. For future studies, a calibration of this factor is recommended taking into account the behaviour of walls described in Chapter 2 is recommended, for different relations between H and L and different dimensions of openings.

In the calculation of the stiffness of the alignments, the bending component has been neglected due to the overall reduced slenderness of the walls in the Central Pavilion. Strictly speaking, global stiffness could have been considered for slender walls and pure shear stiffness for less slender walls. However, considering a shear stiffness for all walls in the building greatly simplifies the calculation and yields favourable results.

For the distribution of the seismic acting force by the reinforced concrete columns it is necessary to take into account the boundary conditions of each column. In some cases the top of the column lies at the border between walls and reinforced concrete slabs, which have been modelled so that no bending moment is transferred between the wall and the slab as described in 4. Thus, the stiffnesses of the columns are given by equations (18) and (19).

$$K^{c}_{corte,i} = \frac{3. E_b I}{L^3} \tag{18}$$

$$K^{c}_{corte,i} = \frac{12. E_b I}{L^3} \tag{19}$$

Equation (18) is used for a cantilevered column and equation (19) is considered for a fixed-fixed column.

The vertical force of the building is distributed in a similar way to that described for the global analysis, which is by considering areas of influence of the alignments. The areas of influence for this building were defined by plastic yield lines at 45 degrees in the general case. However, one should consider the case of walls with reinforced concrete columns embedded in the wall. For these cases, an axial stiffness distribution is proposed, which in practice results in plastic yield lines with angles greater than 45 degrees, with walls with embedded reinforced concrete columns attracting a greater proportion of vertical force.

The chart shown in Figure 8 compares the results obtained in both the numerical model and the ICIST/ACSS methodology variant for floor z=-3.1m in the x direction.

Figure 7: Bar chart showcasing comparison between results obtained from the numerical model with the ICIST/ACSS methodology variant, for an analysis by alignment

Although it is not possible to show the full results in the present paper, these values are considered representative of the overall analysis by the ICIST/ACSS methodology variant.

6. Conclusions and recommendations for future studies

The application of the ICIST/ACSS methodology variant to the Central Pavilion allowed for satisfactory results with regards to a global analysis, being generally on the conservative side. The values obtained for F_{Sd} at a global level are 10 to 20% higher than those obtained through the numerical model, indicating very close and conservative results for a modal participation factor of 0.85 as recommended by Eurocode 8. On the resistance end, the values obtained for F_{Rd}, considering the distribution of the vertical force by influence areas and axial stiffness of the resisting elements, are 2 to 7% higher than the values obtained through the numerical model. Comparing the safety factors (SF), the values obtained through the ICIST/ACSS methodology variant fall between 85.18% and 96.59% of the results obtained by the numerical model, very close values and on the conservative side. Regarding the alignment analysis, the obtained results diverge considerably from those obtained through the numerical model in some cases. In general terms, the results of security verification are smaller than those of the numerical model, which indicates an analysis on the conservative side. The distribution of the seismic forces by the various alignments through the shear stiffness with the application of the stiffness reducing factor f due to wall openings was the method that generated better results when compared to a global stiffness distribution or cross-sectional area. However, for future studies it is recommended to calibrate the reducing factor f in order to better consider different dimensions and arrangements of openings in masonry walls, as well as the interaction between masonry walls and reinforced concrete columns embedded in the walls. The application of the variant of the ICIST/ACSS methodology in the Central Pavilion of the Instituto Superior Técnico proved to be a considerable challenge due to its high irregularity in both plan and elevation, however the assumptions considered throughout this paper allow obtaining results that can be considered representative of an analysis performed through a numerical model.

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