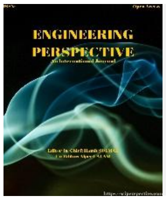

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Research Paper

Analysis of the Seismic Vulnerability of a 3-Story Building, Using Typical Structural Sizing Methods Used In Mozambique

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ABSTRACT

This study examines the seismic vulnerability of reinforced concrete structures in Mozambique, focusing on the effects of seismic action on structural performance and safety. Due to a lack of seismic-specific regulations in Mozambique, many structures are built without considering earthquake resilience. This research evaluates a reinforced concrete building's response to seismic forces using modal analysis by response spectrum, guided by Eurocode 8 standards. The study begins with an overview of Mozambique's seismic history, geological features, and high-risk regions. It also explores structural design practices in the context of Mozambique's limited regulatory framework (REBAP/RSA). Using Robot Structural Analysis Professional software (2020), we conduct a modal analysis to assess structural behavior beyond the elastic range. Our results indicate maximum displacements of 1.7–1.9 cm and basal shear forces of 1,423–1,282 kN in the X and Y directions, respectively. Floor drift ranges from 0.28 to 0.83, with modal data showing that the first mode, dominated by torsion (86.06%), compromises seismic resilience. The second mode exhibits a translation in the X direction with a modal participation of 96.83%, while the third mode shows a translation in the Y direction with a modal participation of 98.74%. These findings imply the structure lacks adequate torsional resistance, potentially endangering its seismic integrity. Future research should explore model variations and analyze structural responses on different types of soil. Investigating optimized building designs to meet seismic demands in Mozambique and globally could improve safety in earthquake-prone areas.

Keywords: Linear analysis; Seismic analysis; Dynamics analysis.

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1. Introduction

1.1. Background

Mozambique has experienced over 256 earthquakes with magnitudes exceeding 2.5 since 1973. Active seismicity across Mozambique, particularly in the central and northern regions seismic events have caused considerable damage, including the destruction of approximately 300 residential units. The most significant earthquake happened on February 22, 2006, with a magnitude of 7.0 on the Richter's scale, which caused 5 deaths, 28 injuries, destruction of more than 280 houses, and more damages. This earthquake underscored the vulnerability of existing structures, 160 shops, and five schools in Espungabera, Beira, and Chimoio.

To address these challenges, civil engineering professionals in Mozambique often adopt international standards for structural design to quantify the effects of seismic activity. The most commonly used standard is Eurocode 8, which establishes general guidelines for designing earthquake-resistant structures, including performance requirements, compliance criteria, and safety verification for limit states. Eurocode 8 also covers geotechnical factors, such as ground conditions and seismic action, acknowledging that the interaction between soil and structure is fundamental to a structure's seismic performance. Understanding soil properties, including parameters for resistance, stiffness, and damping, is therefore essential. This study includes parametric analysis, a historical overview of seismic activity in Mozambique, and geological characterization, aiming to identify proximity to active faults and determine ground type to define seismic action parameters accurately

1.2. Seismic Analysis

1.2.1. Modal analysis by response spectrum

This analysis method is applicable when a structure's geometry does not satisfy the requirements of the lateral force analysis method. It enables seismic analysis of structures with linear behaviour, providing insights into time-dependent responses, particularly the maximum response values. Therefore, all responses from vibration modes that significantly impact the overall structural response must be considered. A mode's contribution is considered significant if the sum of its effective modal masses exceeds 90% of the total mass of the structure and each modal mass is greater than 5% of the total mass. If these conditions are not met, the required number of vibration modes can be determined using the following formulas in Eq. (1) and Eq. (2):

$$k \geq 3 \times \sqrt{n} \tag{1}$$

$$T_k \leq 0.20s \tag{2}$$

Where:

n – represents the number of floors above the foundation;

k – represents the number of modes considered;

T_k – represents the period of mode k.

According to [1], two vibrations are considered independent, while combining modal responses, if the period of the subsequent mode is equal to or less than 90% of the preceding mode's period. If this condition is unmet, a more complex approach, such as the complete quadratic combination (CQC), should be applied.

In cases where a spatial analysis model is adopted, the effects of accidental torsion must also be considered, as they introduce torque moments within the structure. Eurocode 8 (EC8) recommends calculating these moments are calculated using Eq. (3):

$$M_{ai} = e_{ai} \times F_i \tag{3}$$

Where:

M_{ai} – represents the torque moment of the vertical axis applied to the floor i;

and $_{ai}$ – represents the accidental eccentricity of the mass on the floor i;

F_i – represents the horizontal force applied to floor i, in all directions.

The M_{ai} value must be defined taking into account both signs, positive and negative, in order to properly characterize the effect of the eccentricity of the masses on the structure. The seismic combination should be considered using the following Eq. (4) and Eq. (5):

$$E_{EDX} \text{ "+" } 0.30 \times E_{EDY} \tag{4}$$

$$E_{EDX} \times 0.30 \text{ "+" } E_{EDY} \tag{5}$$

Where:

E_{EDX} – represents the stresses resulting from the application of seismic action as a function of the horizontal x-axis of the structure;

E_{EDY} – represents the stresses resulting from the application of seismic action as a function of the orthogonal horizontal y-axis of the structure.

1.3. Mozambican Seismic Context

According to [2], concerns about seismic risk in Mozambique were quite low, until the earthquakes of February 2006 which accentuated the reality of the risk. Although the prioritization of seismic risk assessment in Mozambique remains open to debate amidst various competing issues, it would be imprudent for professionals in the construction industry to continue designing and building structures without accounting for earthquake resilience.

1.3.1. Regions Prone to Seismic Events in Mozambique

Seismically active areas in Mozambique are located primarily along the African Rift Valley and the Mozambique Channel. The Rift Valley divides near Lake Victoria, located between Kenya, Tanzania, and Uganda, forming two branches: the Western branch, which includes Lakes Tanganyika and Niassa and terminates in central Mozambique, and the Eastern branch, which extends along Kenya and ends in southern Tanzania. The difference in separation speeds within the Rift partly explains why seismic activity in Mozambique's southern Rift areas is less intense and frequent than in the northern regions. However, USGS data indicates that Mozambique has experienced consistent seismic activity over the past 33 years, with most of Southern Africa's seismic activity associated with the East African Rift system.

It is also important to note that cities like Maputo and Beira face an added risk from tidal waves, or tsunamis, which may result from seismic or underwater disturbances as presented in Fig. 1. Provinces within this zone should therefore be prepared for these additional potential hazards [3].

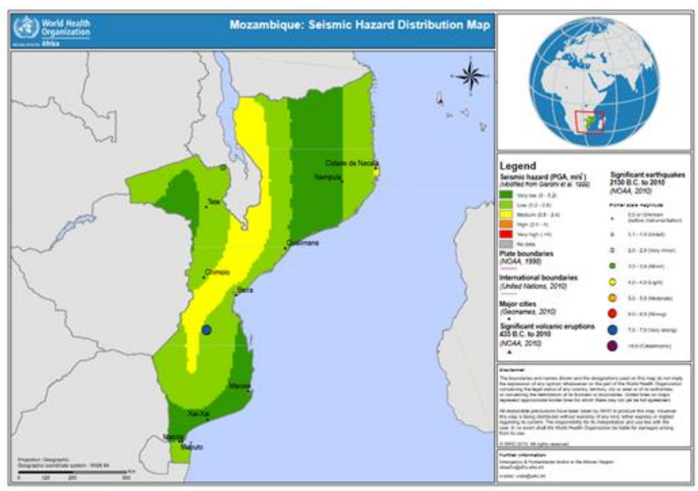

Fig 1. Seismic risk map of Mozambique as per WHO. Dark green: very low risk (0 – 0.2 m/s ^{2}), light green: low risk (0.2 – 0.8 m/s ^{2}), yellow: medium risk (0.8 – 2.4 m/s ^{2}). The blue circle represents the occurrence of an earthquake on a scale of 7.0 to 7.9 MW (Source: adapted from the WHO by the Author, 2024)

1.3. Historical Context of Mozambican Seismic Events

Before 1973, three major events had been recorded. The first was on May 10, 1951, with a magnitude of 6.0 and an epicentre approximately 100 km from Beira. The second and third occurred on July 20 and September 20, 1957, respectively, around 200 km from Chimoio and Beira; both recorded at the JOH station in Johannesburg with a magnitude of 6.0 [3].

Since 1973, data from the US Geological Survey [4] show that around 256 earthquakes with magnitudes greater than 2.5 have occurred throughout Mozambique. Notably, over 87.5% of these had magnitudes above 4.0, with at least 25 earthquakes reaching 5.0 or higher—generally considered the minimum on the Richter scale for plate movements capable of causing structural damage. The majority of seismic activity in Mozambique is classified as shallow, with 210 recorded earthquakes originating at depths less than 10 km, while only 17.97% of events had foci deeper than 33.3 km. Most of these earthquakes were attributed to normal or strike-slip faults as illustrated in Fig. 2. Moreover, according to the [4], the largest earthquake recorded in the Rift Valley since 1900 reached a magnitude of 7.0, with an epicentre in the Machaze district of Manica province at an approximate depth of 10 km.

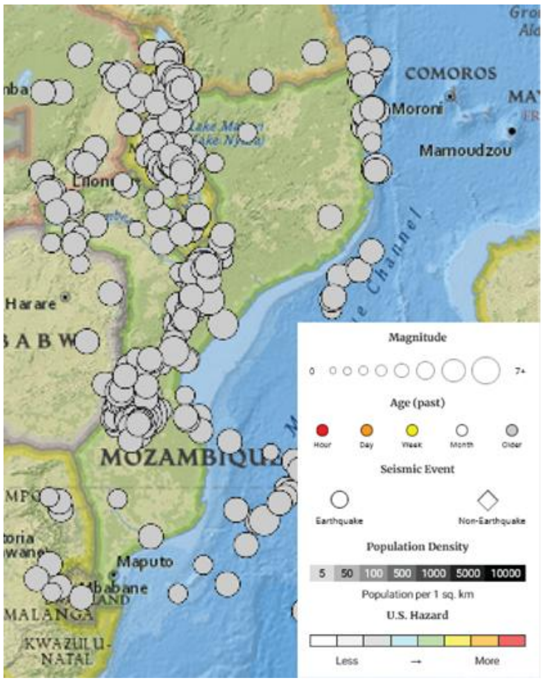

Fig 2. Distribution of seismic activity and representation of epicenters in Mozambican territory, between 1973 and February 2024 (Source: adapted from the USGS by the Author, 2024)

Based on data provided by [4], there has been an increase in seismic activity in the central area of Mozambique since February 2006. This phenomenon has been so frequent that approximately 37.5% of the earthquakes recorded by the USGS since 1973 were observed throughout 2006. Most of the epicentres of these seismic phenomena are located in the Machaze district, in the province of Manica.

- **Largest Earthquake Occurred in Mozambique**

The largest earthquake recorded in Mozambique occurred on February 22, 2006, in Machaze, Manica province, with a magnitude of 7.0 as shown in Fig. 3. According to [3], this earthquake damaged at least 160 buildings in Espungabera, Beira, and Chimoio, as well as the water supply system in Chitobe. Approximately 300 homes, 8 shops, and 5 schools were affected, along with the degradation of the Inhambane and Maxixe jetties, necessitating emergency intervention. [2] noted that the Machaze earthquake to have been caused by the rupture of a previously unidentified fault, which presumably might have been an ancient, slow-moving fault or a new structure associated with the southern propagation of the Rift.

The occurrence of a high-magnitude earthquake in an unexpected location underscores the need for thorough seismic vulnerability assessments in Mozambique, as there may be additional, unidentified faults near critical infrastructure. Considering these factors is essential during the structural design phase to account for potential earthquake impacts across the country.

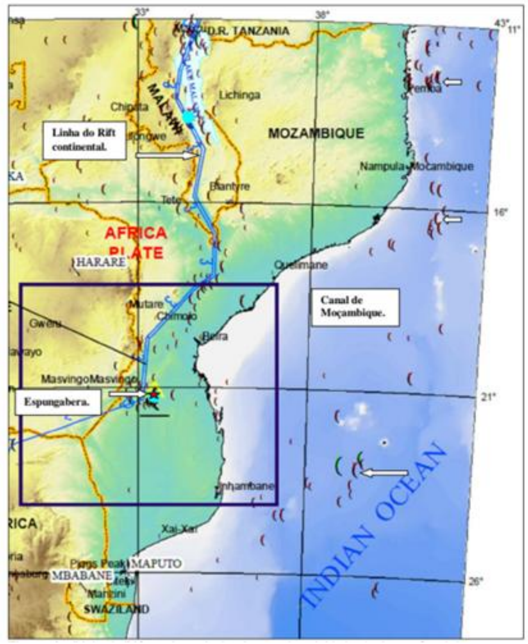

Fig 3. Machaze Earthquake Geographical Location (Source: adapted from the USGS by Author, 2024)

1.4. Mozambican Geological Context

Mozambique is located on the eastern margin of the African Plate, at the southern end of the East African Rift, which delimits the two parts of the African plate that separates it, the Nubian Plate and the Somali Plate, and extends from the Gulf of Aden in the north to the south of Mozambique, for more than 3,000 km [5].

[6] estimated that the Eastern branch of the Rift extends southwards to the Southwest Indian Ridge and that the degree of separation of the Rift is estimated at around 8.3 ± 1.9 mm/year in the Gulf of Aden region and 3.6 ± 0.5 mm/year in the intersection zone of the Eastern branch of the Rift with the ridge, due to persistent seismicity along the Mozambique Channel as illustrated in Fig. 4. Such extensions along the Rift began around 45 million years ago. The first faults around emerged 30 Ma ago in Ethiopia and propagated southwards, with the first faults at the southern end of the Western branch of the Rift emerging around 10 Ma ago [3].

The Rift is moderately developed in the North and Center of the country, where Lakes Niassa and Chirua are located. [2] describes the geology of Mozambique in two-thirds of the country's surface, consisting of crystalline rocks older than 570 Ma, Precambrian terrains, in which tectonic elements resulting from collision processes between plates predominate.

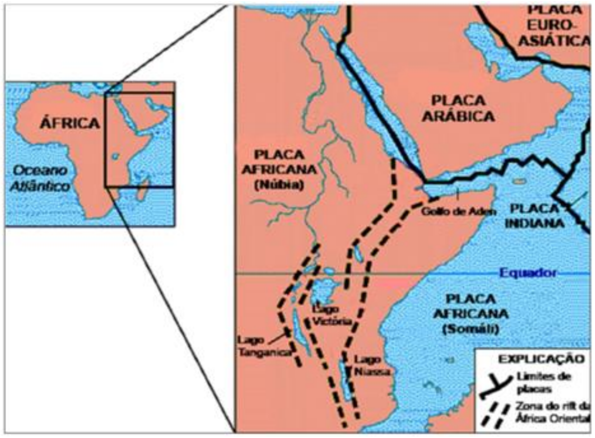

Fig 4. The complex of tectonic faults in East Africa. (Source: adapted from Fonseca, 2010)

This is shown by the occurrence of plateaus configured mainly in the Central and Northern regions of the country, in detail, the plateaus of Niassa, Mueda, Chimoio, and Angónia, as well as the main mountainous formations consisting predominantly of Chimanimani Massif, Maniamba Mountain Range, and the Chire-Namuli Formations. [2], also states that "the remaining surface is made up of sedimentary rocks less than 570 Ma old, Phanerozoic terrains, in which tectonic elements resulting from the opening of the Indian Ocean, associated with the break-up of the supercontinent Gondwana, and tectonic elements resulting from the advance of the Rift are found".

1.5. Mozambique Seismic Monitoring Infrastructure

Until the 12th of February 2024, Mozambique did not have an independent seismic data analysis and processing centre. Only after the inauguration of the Geology laboratory in the KaMubukuana Municipal District in Maputo City did Mozambique become capable of identifying earthquakes in real time and accurately and efficiently determining the coordinates of earthquake epicentres. This is why seismic event data was made available on the USGS website, which was the fundamental source of data on the date of occurrence, location, magnitude, duration, and depth of earthquakes in Mozambique. It should be noted that this research only includes information on events from 1973 onwards.

1.6. Review of National Regulations

The fact that seismic risk is partially reduced in Mozambican territory the various priorities that occurred in the country during the years of civil war and the presence of other more devastating natural disasters, culminated in the lack of specific regulations for seismic dimensioning in Mozambique.

1.6.1. Limitations of the Existing Regulations

According to the information proclaimed by the Ministry of Public Works and Housing (MOPH) of the Republic of Mozambique cited by [7], "the regulation in force for the dimensioning of reinforced concrete structures is the Regulation for Reinforced and Prestressed Concrete Structures (REBAP, 1983), and for the quantification of static actions it is the Regulation for Safety and Actions for Building and Bridge Structures (RSA, 1983)". It should be noted that the referenced regulations are full versions of the Portuguese regulations, and no adaptation has been made, especially for the quantification of actions for the Mozambican context. Although the REBAP presents implicit construction provisions that consider the effects of seismic action, the quantification of this action, as set out in the RSA for Portugal, has been disregarded in structural design in Mozambique.

According to [8], the development of the RSA and REBAP has been in effect for some time now, which increases the need for a review and respective update. Furthermore, although these regulations present some concepts similar to EC8, defined for the improved ductility class, they do not make them applicable conditions in practice. Because these regulations advocate a design in which the structure presents a linear behavior that aims to resist seismic action fundamentally through the resistant capacity of the structural elements in an elastic regime.

EC8 suggests a new philosophy called "Capacity Design", which allows for non-linear analyses to be carried out, both static (pushover) and dynamic. This type of analysis allows for a more realistic assessment of the behavior of structures, representing their response when subjected to seismic actions more reliably. Although the results of the survey conducted for this research do not represent the universe of civil engineering professionals in Mozambique, some consider that the use of RSA and REBAP are essentially conservative in nature, since they lead to the oversizing of structures, which may provide greater lateral resistance when subjected to seismic excitations.

1.6.2. Differences Between the Existing Regulation and Eurocode 8

According to [8] aspect that denotes the need for a review of the RSA, "is the fact that it does not refer to any verification associated with the limitation of damages, one of the major differences between the two regulations, thus highlighting the importance that EC8 gives to the limitation of economic losses". It is also worth noting that in the RSA, seismic action is considered a variable action, and therefore has a probability of exceeding 5% in 50 years. Therefore, we are faced with a disparity between a return period of 475 years defined in EC8 and the 975 years established in the RSA. Concerning seismic zoning, enormous changes arise in EC8, compared to that established in the RSA, decimating the lack of coherence of the RSA, whose zoning is unique and developed according to distant seismic action, since, depending on whether distant seismic action or close seismic action is considered, there will be different epicentre positions.

Concerning the types of ground conditions, it is also worth noting the greater rigour on the part of EC8, which considers five types of ground conditions, unlike the RSA, which establishes three types of ground conditions. The aim is therefore to achieve a more defined and coherent classification, which is consistent with the values defined in the response spectra, which show considerable differences in spectral acceleration depending on the type of soil in question, thus justifying the need for a more cautious and demanding discretization of the different types of terrain. Therefore, regarding the representation of seismic action, both present two types of seismic action, in detail, the moderate magnitude earthquake at a small focal distance and the greater magnitude earthquake at a greater focal distance. Regarding ductility classes, a similar scenario is noted between structures with the low ductility class in EC8 and the Normal Ductility structures recommended in REBAP. "As for the Medium and High ductility classes of EC8, there is a significant difference to REBAP in that it only presents an additional ductility class called Improved Ductility. Therefore, there is no direct relationship between classes, however, Improved Ductility structures are similar to Medium Ductility structures (MCD)" [8].

Given these and other aspects, and so that the effect of seismic action is precisely defined and quantified in the best possible way, Eurocode 8 was drawn up, which emerges as a European regulatory standard for structural dimensioning in the face of seismic action, and which replaces part of the regulations relating to seismic action that apply to Portugal, namely RSA and REBAP, which, having been drawn up more than thirty-seven years ago, are now outdated and have some omissions that need to be filled.

1.6.3. Safety Verification of Structural Elements based on EC8

EC8 and EC2 provide important rules for verifying safety, relating to beams and columns. In this context, an approach is made to the geometric constraints, the quantification of sizing actions and the verification of the ultimate limit state.

2. Materials and modeling

2.1. Characterization of the Building

The building considered in this research is intended for institutional use as shown in Fig. 5. It consists of 3 floors with a ceiling height of 3.00 m, in a reinforced concrete frame of footings, piles, rectangular pillars of (0.20x0.80) m^2, (0.20x1.00) m^2 and circular ones with a radius of 0.30 m and 0.35 m, beams of (0.20x0.50) m^2, (0.20x0.60) m^2 and (0.20x0.70) m^2 and slabs with a thickness of 20 cm and 22 cm. All floors are intended to serve the offices and services related to UNICEF activities. The building has a total area of 818.00 m^2, distributed by floor according to the Table

1.Fig 5. Main facade of the building under study (Source: adapted from ArchiCad 24.0.0 INT Component by Author, 2024)

Table 1. Description of the building areas (Source: adapted from project by Author, 2024)

Floor	Gross Area (m^2)	Height (m)	Function
Ground floor	261.80	3.00	Office
1st Floor	277.55	3.00	Office
2nd floor	278.65	3.00	Office
Terrace	108.75		Warehouse
Total	926.75	9.00	

The data in Table 1 were obtained from the architectonic project of the building, where the area of each individual floor was calculated due to the building's varying geometric forms across different floor. Since it is a private building, we must protect the interests of the owners. Therefore, readers who require detailed access to the floor plans should contact the authors.

2.2. Geographical Location

The building under study, namely the UNICEF Offices, is located in Maputo City, at Avenida do Zimbabwe, No. 1422/1440, Sommerschield Burgh, Plot No. 141B/399 in the District of Sommerschield, Kampfumo, on a regular plot of land with plan dimensions of 18x78, and with the following coordinates (Lat: **-25.954963**, Long: **32.59 4350**) as presented in Fig. 6.

Fig 6. Main facade of the building under study (Source: adapted from ArchiCad 24.0.0 INT Component by Author, 2024)

2.3. Structural Materials

In order to ensure the building's resistance to seismic action in plastic regimes, the structure is composed of reinforced concrete, resistance class C25/30, and A400 NR SD steel as indicated in Table 2.

Table 2. Characteristics of the concrete and steel adopted

Concrete C25/30	Steel A400 NR SD
$f_{ck} = 25.0$ MPa	$f_{yk} = 400$ MPa
$f_{cd} = 16.7$ MPa	$f_{yd} = 348$ MPa
$f_{ctm} = 2.6$ MPa	Es = 200 GPa
$E_{cm} = 31.0$ GPa	$\gamma_s = 78.5$ kN/m^3
$\gamma_c = 25.0$ kN/m^3	

2.4. Geotechnical Conditions

Based on the variability like the soil, a geotechnical assessment carried out by [9] indicates that the building has deep foundations consisting of bored concrete piles with a diameter of 0.60 m and a length of 18 m and 20 m to take into account the expected settlements and the bearing capacity of the soil. Due to the low bearing resistance of the soil, the length of the pile was dimensioned considering the friction resistance, limited to 75 kPa. The transition between the piles and the pillars is made by concrete blocks with a height of 0.80 m, joined by earth beams with sections ranging from (0.35x0.80) m^2 to (0.30x0.60) m^2.

2.5. Acting Actions

An action represents any agent capable of producing significant states of tension or deformation in any structural element (Testino, 2023) are presented in Table 3.

2.5.1. Permanent actions

Table 3 summarizes the permanent actions considered in the structural analysis, including the self-weight of materials such as reinforced concrete and steel, as well as additional loads from slab coverings and masonry walls.

Table 3. Quantification of permanent actions (Source: adapted by Author, 2024)

Permanent Actions	Load
DL Reinforced Concrete	25.0 kN/m^3
DL Steel	78.5 kN/m^3
SIL Slab Covering (Ground floor, 1st floor)	1.2 kN/m^2
SIL Slab Covering (2nd floor)	2.0 kN/m^2
SIL Slab Covering (Terrace)	1.5 kN/m^2
SIL Internal masonry wall	1.5 kN/m^2
SIL External masonry wall	6.4 kN/m^2

2.5.2. Variable Actions

The building belongs to categories B and C5, relating to the office and warehouse areas, with the presence of a non-accessible roof of category H as illustrated in Table 4.

Table 4. Values of overloads and combination coefficients (Source: adapted from EC1 and EN 1990 by Author, 2024)

Category of areas	Category	q_k (kN/m^2)	Coefficients		
			Ψ_0	Ψ_1	Ψ_2
Floors	B	3.0	0.7	0.5	0.3
	C5	5.0	0.7	0.7	0.6
Non-accessible roof	H	1.0	0	0	0

Where:
q_k - Value of uniformly distributed overload;
Ψ_0, Ψ_1, Ψ_2 - Combination coefficients.

2.5.3. Thermal Actions

According to EC2, it is not necessary to consider the effects of thermal action, since the building's dimensions in the plan are less than 30 m.

2.5.4. Wind Action (WD)

Based on EC1, the parameters adopted to quantify the effects of wind action are as given in Table 5:

Table 5. Parameters adopted to quantify wind action according to EC1, 1991

Zone Classification	Zone B	v_b = 30 m/s
Terrain Category	Type II	z_0 = 0.05 m; z_{min} = 3.0m
Peak dynamic pressure q_p (z)		q_p (z) = 1.23 kN / m^2

In any case, it was found that wind action is less conditioning when compared to seismic action.

2.5.5. Seismic Action (SE)

To characterize the seismic action in the analysis of the building under study, the fundamental principles of EC8 were considered. As such, the definition of seismic action is based on acceleration response spectra, which reveal the seismic movement on the ground surface, having two quantities, vertical component and horizontal component. However, for this project, only the horizontal seismic action was considered. While designing seismic action, a parametric study of the site must be carried out, with a view to defining the characterization of the seismic zone under study, the typology of the terrain where the structure is located as shown in Table 9, the importance class of the structure as given in Table 7, and the acceleration on the surface [9] as presented in Table 8.

2.5.5.1. Characterization of the Seismic Zone

According to the [11], the study site is located in a low seismic risk zone, with earthquakes that cause surface acceleration of up to: PGA < 0.8 m/s ^{2} as indicated in Table 6.

Table 6. The reference value of maximum acceleration

Region	Seismic Action	agR (m/s ^{2})
Maputo	Type 1	0.8
	Type 2	0.8

2.5.5.2. Terrain Type

The building under study is considered to be located on a Type D terrain, described as a “deposit of non-cohesive soils of low to medium compactness, with or without some strata of soft cohesive soils, or of predominantly cohesive soils of soft to hard consistency” [12].

2.5.5.3. Importance Class

Table 7 presents the importance coefficients (γ_1) used for seismic analysis, based on EC8 guidelines. For Importance Class II buildings, a value of 1.00 is applied for both interplate and intraplate seismic actions.

Table 7. Importance coefficients $\boldsymbol{\gamma}$ 1 (Source: adapted from EC8 by the author, 2024).

Importance Class	Seismic Action Type 1 (Interplate)	Seismic Action Type 2 (Intraplate)	
II	1.00	1.00	1.00

2.5.5.4. Surface Acceleration

Having defined the reference value of the maximum acceleration and the importance coefficient, it becomes possible to define the value of the surface acceleration through the following expression in Eq. (6):

$$A_g = \gamma 1 \times A_{gR} \tag{6}$$

Where:
A_g – calculation value of the surface acceleration;
γ1 – importance coefficient;
A_{gR} – represents the reference value of the maximum acceleration of the ground.

Table 8. Surface acceleration values (Source: adapted from EC8 by the Author, 2024).

Region	Seismic Action	AgR (m/s ^{2})	$\boldsymbol{\gamma}$ 1	Ag (m/s ^{2})
Maputo	Type 1	0.8	1.0	0.8
	Type 2	0.8	1.0	0.8

2.5.5.5. Soil Coefficient

Assuming that the surface acceleration value is Ag ≤ 1 m/s ^{2}, the soil coefficient is defined based on the following expression in Eq. (7):

$$S = S_{max} \tag{7}$$

2.5.5.6. Other Parameters for Characterizing Seismic Action

Table 9 shows the values of key parameters for the elastic response spectrum on terrain type D, based on EC8. The parameters vary depending on whether the seismic action is Type I (interplate) or Type II (intraplate), with notable differences in the period values.

Table 9. Values of the remaining parameters of the elastic response spectrum for action Type I and II seismic (Source: adapted from EC8 by the Author, 2024)

Terrain Type	S_{max}	T_B (s)	T_C (s)	T_D (s)
D (Type I Earthquake)	2.0	0.1	0.8	2.0
D (Type II Earthquake)	2.0	0.1	0.3	2.0

2.5.5.7. Horizontal Elastic Response Spectrum of Acceleration

With the characterization factors of the seismic action obtained, the horizontal elastic response spectrum of acceleration Se (T) was defined, which represents the horizontal seismic action of the ground movement as shown in Fig. 7 and Fig. 8. This spectrum is determined by [12] according to the following expression Eq. (8):

$$T_c \leq T \leq T_D : S_e(T) = A_g \times S \times \eta \times 2{,}5 \times \left[\frac{T_C}{T}\right] \tag{8}$$

Where:

$S_e(T)$ – elastic response spectrum;

T – vibration period of a linear system with one degree of freedom;

A_g – design value of the acceleration at the surface;

T_B – represents the value of the lower limit of the spectral acceleration period;

T_C – represents the value of the upper limit of the spectral acceleration period;

T_D – value that defines the beginning of the branch displacement in the response spectrum;

S – represents the soil coefficient;

η – represents the damping correction coefficient, with the reference value η = 1 for 5% viscous damping.

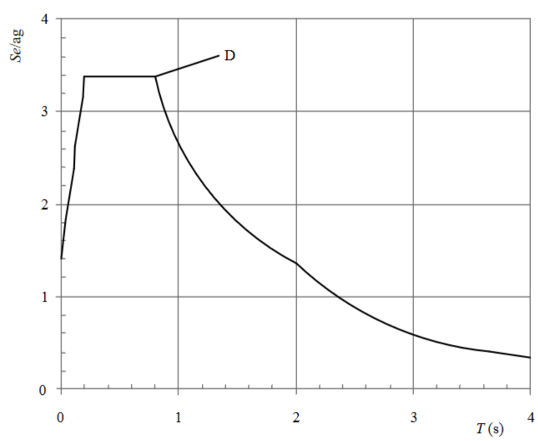

Fig 7. Type I elastic response spectrum recommended for type D ground with 5% damping (Source: adapted from EC8 by the Author, 2024)

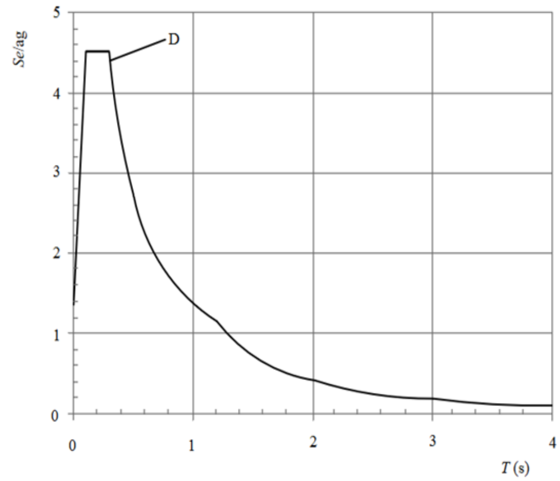

Fig 8. Type II elastic response spectrum recommended for type D ground with 5% damping (Source: adapted from EC8 by the Author, 2024)

2.6. Action Combinations

The study of action combinations is carried out to study the most unfavourable effects that the structure may be subjected to. These combinations are analyzed for Service Limit States (SLS) and Ultimate Limit States (ULS) are presented in Table 10 and Table 11.

Table 10. Project Action Combinations (Source: adapted by Author, 2024)

Case	Analysis Type	Combination
1	ULS	$1.35 \times (DL + SIL)$ $+1.5 \times LL$
2	SLS	$1.00 \times (DL + SIL)$ $+1.0 \times LL$
3	Seismic	$1.00 \times (SEX + SEY)$
4	Seismic X	$1.00 \times (DL + SIL)$+$1.0 \times LL + 1.0 \times SEX$
5	Seismic Y	$1.00 \times (DL + SIL)$+$1.0 \times LL + 1.0 \times SEY$
6	E_{EDX} "+" 0.30 $\times E_{EDY}$	$1.00 \times SEX + 0.30 \times SEY$
7	E_{EDX} $\times$ 0.30 " + " E_{EDY}	$0.30 \times SEX + 1.00 \times SEY$

2.6.1. Partial's Coefficients Security (γ) and coefficients combination (ψ)

Table 11. Partial Coefficients security and adopted coefficients combination (Source: adapted of EC0 by Author, 2024)

Action		Partial's Coefficients (γ)		Coefficients (ψ)		
		Favourable	Unfavorable	Ψ_0	Ψ_1	Ψ_2
Permanent Actions	DL	1.35	1.00	-	-	-
	SIL	1.35	1.00	-	-	-
	LL	1.50	0.00	-	-	-
Variable Actions	TL	1.50	0.00	0.6	0.5	0.0
	IF	1.00	0.00	-	-	-
	WD	1.50	0.00	0.6	0.2	0.0

2.7. Effect of Seismic Action, Modeling and Structural Analysis

To ensure that the objective of the earthquake-resistant design based on EC8 is achieved, two levels of seismic verification arise, namely, the requirement of non-collapse and the requirement of damage limitation. These requirements are fundamental for the structure to have good seismic performance. The complexity of manually calculating structural elements, to obtain a dynamic and elastic structural analysis, demands the use of an automatic calculation tool. Therefore, to carry out this analysis, AutoDesk Robot Structural Analysis Professional 2020 was used, in which all the structural and geometric characteristics of the building were defined, including the acting actions. Based on the created model, a structural analysis was carried out using the appropriate analysis method prescribed by EC8, namely the modal analysis by response spectrum about the consideration of the effect of seismic action.

2.7.1. Effects of Seismic Action

In order to analyze the effects of seismic action on the building under study, the structural characterization of the building was carried out and the regularities in plan and height were checked, with emphasis on the ductility class and the value of the behavior coefficient, to obtain the response spectrum for calculation based on EC8.

2.7.2. Structural Characterization of the Building

Based on the geographical location of the building under study and a preliminary description of the geometric dimensions of the structural elements, a structural characterization was carried out, considering that the building was designed based on the basic principles of EC8, proving useful for ensuring the fundamental requirements of non-occurrence of collapse and damage limitation. From this perspective, the building incorporates the solution of uncoupled reinforced concrete walls that start from the foundations and extend to the terrace, which is characterized by the presence of beams, pillars and walls that support the gravitational loads coming from the solid slab floors.

2.7.2.1. Regularity in Plan

The building's plan shows that the structure, although it appears to be regular in plan, meets the slenderness criterion, in which in the largest direction, the structure is 18.00 m and in the smallest, it is 14.80 m, which results in a slenderness of 1.22, which is less than the 4 recommended in EC8. And, because it has a compact plan, with setbacks that do not affect the rigidity of the floor in the plan. This is not because it does not present symmetry in relation to all dimensions of the building x and y, thus compromising the verification of the regularity criterion in the plan is given as Eq. (9).

$$\lambda = \frac{L_{max}}{L_{min}} = \frac{18{,}00\ m}{15{,}20\ m} = 1{,}18 \leq 4 \tag{9}$$

Where:
L_{max}– largest dimension in plan of the building;
L_{min}– smallest dimension in plan of the building.

2.7.2.2. Regularity in Height

In accordance with EC8, the following checks were carried out for the building under study:

- ✓ The lateral load-bearing systems, in particular the frames and the core (stairwell), are not interrupted from the foundations to the top of the building;
- ✓ The lateral stiffness and mass of each floor show a gradual reduction, with no sudden variations, from the base to the terrace of the building structure;
- ✓ The building does not have set-back floors, so the conditions relating to this type of situation are not required to be checked.

As such, it can be concluded that the building under study is regular in height.

2.7.3. Behavior Coefficient

The behavior coefficient (q) is a parameter used to perform calculations with the aim of reducing the forces obtained in a linear analysis, representing an approximation of the ratio between the seismic forces, in which the structure is in an elastic response regime, with 5 % damping, and the seismic force, with non-linear behavior where showed in Table 12. It should be noted that the behavior coefficient is associated with the material and depends on the regularity in height and plan, the ductility class and the structural system is calculated sing Eq. (10).

$$q = q_0 \times k_w \geq 1{,}5 \tag{10}$$

Where:
q_0– basic value of the behavior coefficient;
k_w– coefficient that reflects the predominant failure mode in structural wall systems.

Table 12. Value of the behavior coefficient in a regular system in height (Source: adapted from EC8 by the Author, 2024)

Structural Type	DCM
Uncoupled wall system	3.0

2.7.3.1. Calculation Response Spectra based on EC8

After calculating the behavior coefficient “q”, a reduced response spectrum was defined in relation to the elastic response spectrum as given by Eq. (11). This reduction allows the evaluation of the energy dissipation capacity of the structure as illustrated in Fig. 9 and Fig.10.

$$T_c \leq T \leq T_D;\ \geq \beta \times A_g : S_d(T) = A_g \times S \times 2{,}5 \times \left[\frac{T_C}{T}\right] \tag{11}$$

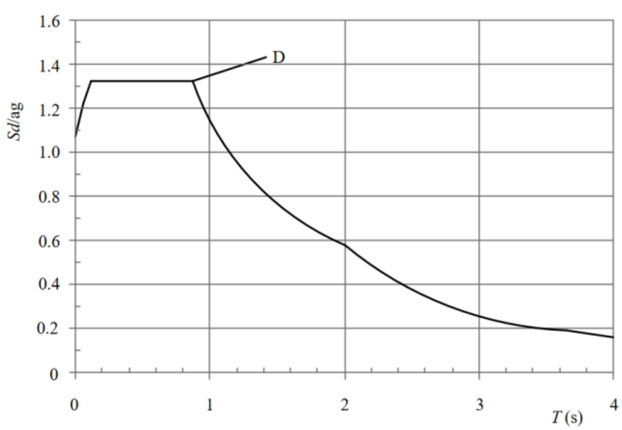

Fig 9. The response spectrum of calculation for Type I seismic action by the behavioral coefficient, q = 3.0 (Source: adapted from MATLAB by the Author, 2024)

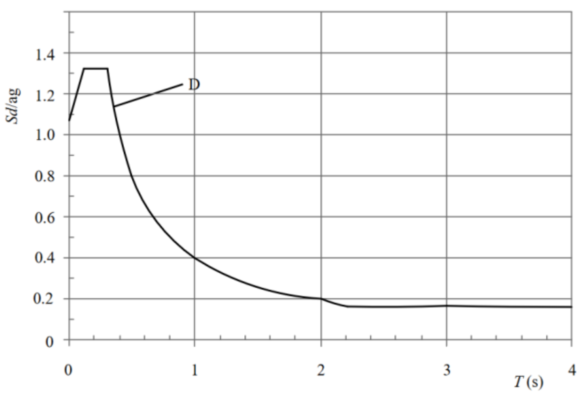

Fig 10. The response spectrum of calculation for Type II seismic action by the behavioral coefficient, q = 3.0 (Source: adapted from MATLAB by the Author, 2024)

2.7.4. Structural Modeling

To model the structure of the building under study, the AutoDesk Robot Structural Analysis Professional software, version 2020, was used, defining the geometric characteristics, the materials in relation to the structural elements and the actions subjected to the structure with the respective calculation combinations.

2.7.4.1. Modeling Methodology

Based on the structural characterization of the building and the classification of the structural system, the building under study sought to follow this process of modelling the different structural elements and the acting actions, making simplifications possible. Therefore, to obtain the structural model of the building and determine the forces to which it is subject, the modelling was carried out according to the following methodology:

- ✓ Definition of the geometry as illustrated in Fig. 11;
- ✓ Definition of the materials and structural elements;
- ✓ Definition of the loads and combination of the actions.

2.7.4.2. Definition of Geometry

In Robot Structural Analysis Professional 2020, the design of the structure's geometry is carried out by defining a structural mesh consisting of a three-dimensional grid, made in accordance with the positioning of the structural elements and their respective midlines (structural axes) as presented in Fig. 12. It should be noted that this project is of an existing structure, in which measurements were carried out to obtain a more realistic representation of what exists on the ground, and at the same time the safety terms of the ULS and SLS conditions defined by Eurocodes were followed.

2.7.4.3. Definition of Materials and Structural Elements

To design the structural model in the calculation software, it was essential to define the materials and cross-sections of the respective structural elements. The materials were defined based on the characteristics described a priori for the structural materials.

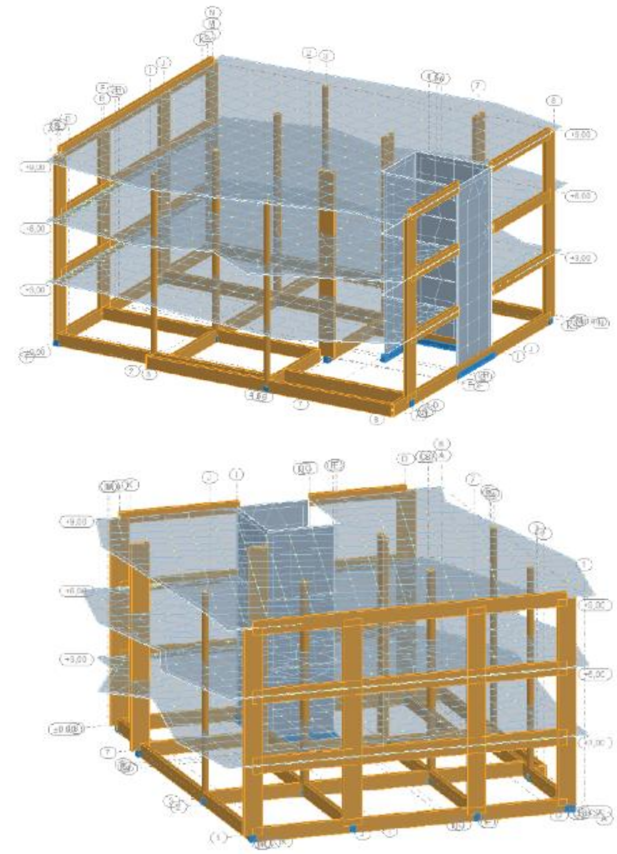

Fig 11.3 D model of the structure in Robot Structural Analysis Professional 2020: Main elevation and Rear elevation, respectively (Source: adapted from Robot by the Author, 2024)

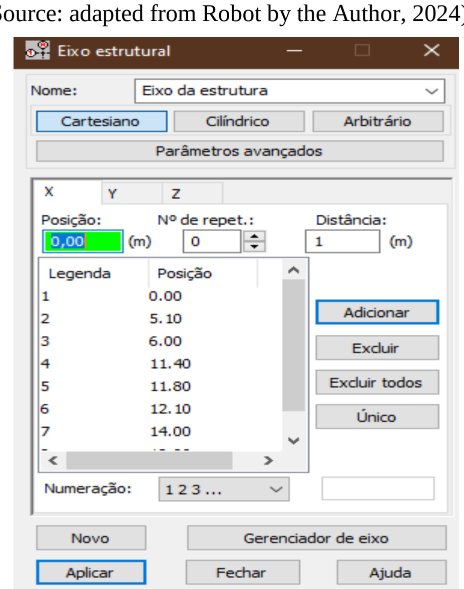

Fig 12. Definition of the structural axis (Source: adapted from Robot by the Author, 2024)

Regarding the elements, two types of structural elements were defined:

- ✓ Bar elements (Frame), which represent the 2-node finite elements;
- ✓ Shell elements (Shell), which represent the 4-node finite elements.

2.7.4.3.1. Columns and beams

The columns and beams were introduced with the help of axes, to guarantee the accuracy of the definition of their coordinates, and were defined in the model as bar elements (Frame) as shown in Fig. 13. Following EC8, and to account for the effect of cracking, the elastic stiffness to bending and shear force was reduced by 50% in both directions of the reinforced concrete bar elements. Regarding the modelling of the beams, their axis was positioned to coincide with the floor level.

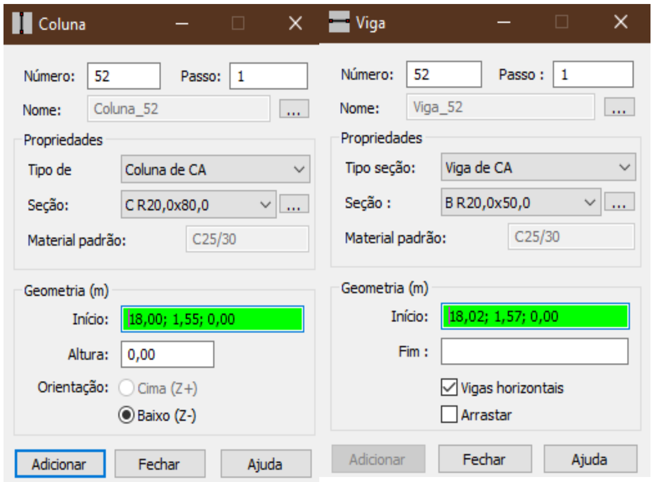

Fig 13. Definition of the columns and beams, respectively (Source: adapted from Robot by the Author, 2024)

2.7.4.3.2. Slabs

The slabs were modelled as shell elements using the Floor option, which allows the representation of homogeneous slabs, taking into account the effects of cross-sectional deformation as presented in Fig. 14. To obtain more accurate results, the slabs were discretized by converting the architectural project into a Robot, to trace the slab contours and also respect the rigid diaphragm condition on the floors.

2.7.4.3.3. Stairs

The stairs were not incorporated into the modelling of the structure of the building under study. To quantify the actions obtained by the existence of the stairs, a gravity shear load of 2.5 kN /m was defined on each floor and the stair core walls.

2.7.4.3.4. Staircase core walls

All the walls of the staircase core were modelled from the shell elements as Illustrated in Fig. 15.

2.7.4.3.5. Definition of Loads and Combination of Actions

The self-weight of the structural elements is considered automatically in Robot Structural Analysis Professional (2020). Meanwhile, the loads of the non-structural elements and the overloads were introduced into the model as surface and linear loads uniformly distributed on the slabs and beams, as applicable, and were defined based on the action table presented above.

To quantify the effect of the seismic action, the calculated response pectra (Sd (T)) were inserted into the model according to EC8 for the two seismic actions related to the Mozambican territory as presented in Fig. 16.

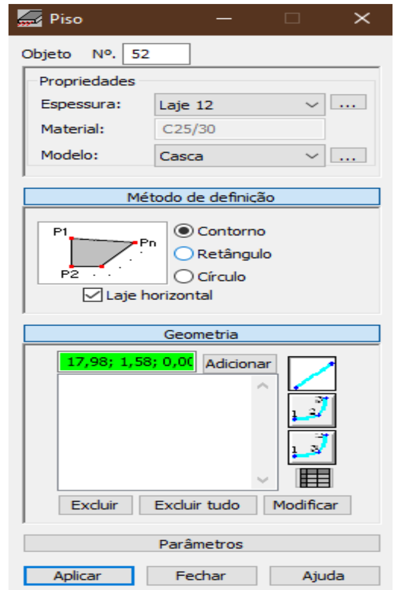

Fig 14. Definition of slabs (Source: adapted from Robot by the Author, 2024)

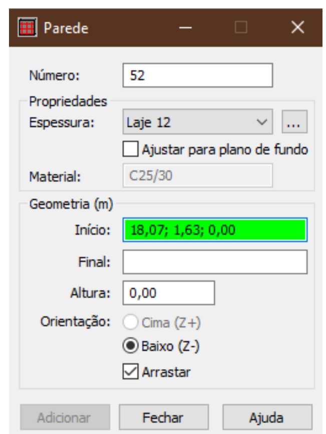

Fig 15. Definition of the walls

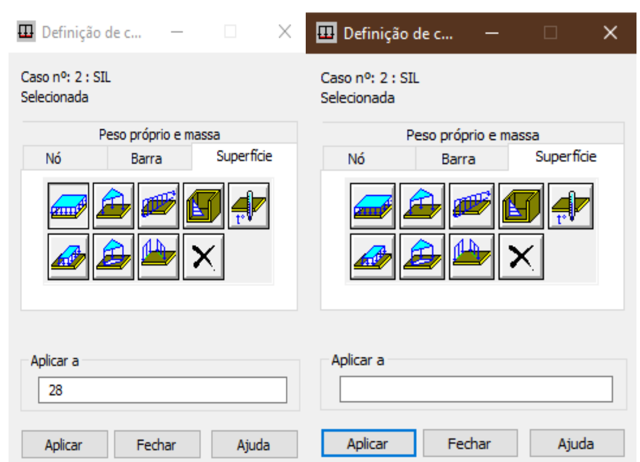

Fig 16. Definition of loads and types of loads

2.7.4.4. Simplifications adopted

The modelling of the building under study addresses three specificities that stand out: the modelling with equivalent sections of the structural elements, the disregard of the soil/structure interaction as illustrated in Fig. 17, since a perfect embedment of the supports at the base is considered, and the modelling of a rigid diaphragm at the level of the floors.

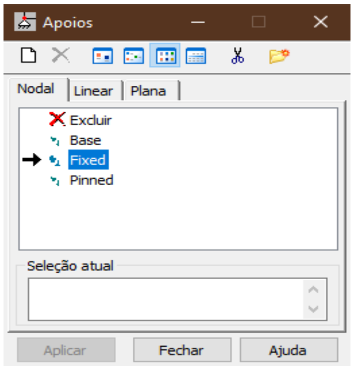

Fig 17. Definition of the embedded supports

2.7.5. Structural analysis

The analysis of the demands is carried out through a 3D spatial calculation, using matrix stiffness methods, considering all the elements that define the structure: columns, walls, beams and slabs. In short, the structure is discretized into bar-type elements, bar mesh and nodes, subject to the action of vertical and horizontal loads. The models are of flat lattice structures, arranged in a rectangular grid that allows the following hypotheses:

- ✓ The structure's behavior is geometrically and physically linear, corroborating the principle of superposition of effects;
- ✓ The floors establish non-deformable diaphragms in their plane;
- ✓ The horizontal forces, which result from the dynamic analysis of the three-dimensional structure, are based on a uniform distribution of the mass across the entire surface of the floors, and act at the level of each floor.
- ✓ The static calculation is performed by solving the following system of linear equations as Eq. (12):

$$[K] \times \{U\} = \{R\} \tag{12}$$

Where:

$[K]$– represents the stiffness matrix;
$\{U\}$– represents the displacement vector;
$\{R\}$ – represents the load vector.

- ✓ The dynamic analysis is performed by solving the following dynamic equilibrium system of equations, which relates the movement of the ground to the response of the structure is calculated using Eq. (13):

$$M\ddot{u} + C\dot{u} + \mathrm{K}u = M\ddot{u}_g \tag{13}$$

Where:

M– represents the mass matrix;
C– represents the damping matrix;
K– represents the stiffness matrix;
$\ddot{u}_g$– represents the ground acceleration;
ű, ů I – represent the acceleration, velocity and displacement of the structure, respectively.

2.7.5.1. Limit State of Cracking

For the situations in which it is intended to verify the cracking limit state, the following methodology is followed. According to [13], the cracking limit state is considered satisfied if the characteristic value of the crack width ω_k, at the level of the most tensioned reinforcement, does not exceed the value of 0.40mm for structural elements with exposure class XC1 and 0.30mm for structural elements belonging to other exposure classes. The characteristic opening crack value, ω_k, is calculated by using the following expression Eq. (14) defined by Eq.(15), Eq.(16), Eq. (17), Eq. (18) and Eq. (19):

$$\omega_k = s_{r,max}(\varepsilon sm - \varepsilon cm) \tag{14}$$

Where:

$$s_{r,max} - \mathrm{k}_3 c + \mathrm{k}_1\mathrm{k}_2\mathrm{k}_4\varphi/\rho_{\mathrm{p,eff}} \tag{15}$$

$$\varepsilon_{\mathrm{sm}} - \varepsilon_{\mathrm{cm}} = \sigma_s - \mathrm{k}_t \times f_{\mathrm{ct,eff}}/\rho_{\mathrm{p,eff}} \times (1 + \alpha_e \times \rho_{\mathrm{p,eff}}))/\mathrm{E}_s \geq 0.60 \times \sigma_s/\mathrm{E}_s \tag{16}$$

$$\alpha_e = \mathrm{E}_s/\mathrm{E}_{cm} \tag{17}$$

$$\rho_{\mathrm{p,eff}} = \mathrm{A}_s/\mathrm{A}_{c,eff} \tag{18}$$

c – longitudinal reinforcement cover;
$k_1 = 0.8$ for high bond bars;
$k_2 = 0.5$ for bending or $= 1.0$ for pure tension; for eccentric tension
$k_2 = (\varepsilon_1 + \varepsilon_2)/(2\varepsilon_1)$; (19)
$k_3 = 3.4$;
k4 = 0.425

2.7.5.2. Limit State of Deformation

The verification of the limit state of deformation is carried out following the Eurocode 2 standard, which specifies that in most structures the deformation for the quasi-permanent combination of actions must be limited to L/250. However, whenever the deformation induced by a certain element of the structure can cause damage to fragile elements, such as masonry walls or coatings, the deformation after the execution of the element or finishing in question must be limited to L/500.

The long-term deformation can be estimated using the following expression Eq. (20):

$$a^{\infty} = a_c(qp) \times (1 + \phi) \tag{20}$$

Where:

a_c (qp) = elastic deformation due to the quasi-permanent combination;

ϕ = creep coefficient (adopted value = 2.00).

So, the value of long-term deformation (mm) for all floors is 9mm ≤ L/500 = 6.40/500 ≈ 13mm.

2.7.5.3. Classification of the structural system

The classification of the structural system was carried out following EC8, considering the uncoupled wall system, with the ductility class DCM, since the resistance to basal shear stress is ensured by most of the columns, which end up being considered as walls, since the following condition is verified h ≥ 4b, and is also guaranteed by the core of the stairs, which is also considered a wall system. The structure of the building under study does not have beams coupling the structural walls, to ensure the lateral resistance of the building through its deformation capacity and a potential for shear rupture, which culminates in the classification of the uncoupled wall system as given by Eq. (21).

$$h \geq 4b$$
$$80 \ \ cm \geq 4 \times 20 \ \ cm \tag{21}$$

Where:

h- represents the length of the column;
b- represents the width of the column.

2.7.5.4. Modal Analysis by Response Spectrum

To carry out the analysis of the structural seismic vulnerability of the building under study, a modal analysis by spectrum was carried out, which makes it possible to obtain the vibration modes of the structure and their respective periods and related frequencies. The period related to each vibration mode gives us the time that the structure takes to perform a complete oscillation, while the frequency shows us the number of complete oscillations performed per second during the occurrence of an earthquake. From the modal analysis by spectrum, the mass participation factors were determined in each direction and for each vibration mode, thus making it possible to analyze the effects of each of the modes on the global response of the structure of the building under study are calculated using Eq. (22) and Eq. (23). Minimum number of modes:

$$k \geq 3 \times \sqrt{n} = 3 \times \sqrt{4} \ \ = \tag{22}$$

Vibration period of the last mode:

$$T_k \leq 0.20\text{s} \tag{23}$$

2.7.5.5. Horizontal seismic forces

The previously defined and represented calculation response spectrum was inserted into Robot Structural Analysis Professional (2020). After applying the response spectra in the respective directions, the value of the basal shear forces acting on the base of the structure in response to the displacements caused by the seismic action was obtained.

“These forces depend, in addition to the earthquake and the components that characterize it in the response spectra, on the masses above the ground, that is, on the gravitational forces that vibrate and also on the fundamental frequency of the building” [3].

2.7.5.6. Accidental Torsion Effects

Through the accidental effects of torsion, it becomes possible to quantify the uncertainty in the location of the masses throughout the useful life of the structure and the spatial variation of the seismic movement. The accidental eccentricity of the center of mass in each floor i can be calculated, which according to EC8 will be displaced by approximately 5% in each direction about its nominal position, according to the following formula in Eq. (24), Eq. (25) and Eq. (26):

$$e_{ai} = \pm 0{,}05 \times L_i \tag{24}$$
$$e_{ax} = \pm 0{,}05 \times L_x = \pm 0{,}05 \times 18 \ \ m = 0.90 \ \ m \tag{25}$$
$$e_{ay} = \pm 0{,}05 \times L_9 = \pm 0{,}05 \times 15.2 \ \ m = 0.76 \ \ m \tag{26}$$

Where:

e_{ai}– represents the accidental eccentricity of the mass of floor i;
L_i – represents the dimension of the floor in the direction perpendicular to the direction of the seismic action.

Based on EC8, the accidental effects of torsion are defined as the envelope of the effects resulting from the application of static loads composed of sets of torsional moments, around the vertical axis z, subjected to each floor i. From the following expression, the torsional moments can be calculated in both directions X and Y using Eq. (27), Eq. (28) and Eq. (29):

$$M_{ai} = e_{ai} \times F_i \tag{27}$$
$$M_{ax} = 0.90 \ \ m \times 1423 \ kN = 1\ 280{,}7\, kNm \tag{28}$$
$$M_{ay} = 0.76 \ \ m \times 1282 \ kN = 974{,}32 \ kNm \tag{29}$$

Where:

M_{ai}– represents the torsional moment of the vertical axis z, applied to the floor i, in kNm;
e_{ai}– represents the accidental eccentricity of the mass of floor i, for all directions X and Y considered, in meters;
F_i – represents the horizontal force acting on the floor i, determined for the X and Y directions, in kN.

2.7.5.7. Calculation of Maximum Displacements of the Structure

With the help of the automatic calculation program, Robot Structural Analysis Professional (2020), the maximum displacements of the structure in the x and y directions were calculated, considering that the structure is subjected to complete quadratic combinations of modal displacements to obtain the final value of seismic actions. Thus, the displacements are obtained from the following equation Eq. (30):

$$U_{Final} = U_{projecto} \times q \tag{30}$$

Where:

U_{Final}– represents the final displacement of the structure in the linear analysis;
$U_{projecto}$– represents the displacement obtained from the linear analysis from the application of the calculation response spectrum.

2.7.5.8. Calculation of Relative Displacements between Floors (Drift)

The evaluation of the relative displacements between floors is carried out directly in Robot Structural Analysis Professional (2020). Damage limitation control is carried out following EC8, which defines that for buildings with non-structural elements made of fragile materials fixed to the structure, the displacement between floors (dr) must be limited to using Eq. (31):

$$d_r = 0.005 \frac{h}{\nu} \tag{31}$$

Where:

h– represents the height between floors;

ν– represents the reduction coefficient equal to 0.40 for type I earthquakes and 0.55 for type II earthquakes.

2.7.5.9. Second-Order Effects

Second-order effects, when compared with first-order effects resulting from the actions and geometric irregularities of the structure, represent the additional effects resulting from the deformation of the structure. Therefore, it is imperative to check the value of the sensitivity coefficient to relative displacement between floors (θ), and based on EC8, second-order effects are not considered if the following condition given in Eq. (32) is met on all floors:

$$\Theta = \frac{P_{tot} \times d_r}{V_{tot} \times h} \leq 0{,}10 \tag{32}$$

Where:

ϴ– represents the sensitivity coefficient to relative displacement between floors;

P_{tot} – represents the total gravity load from all floors above the floor considered, including this one, in the seismic design situation;

d_r– represents the design value of the relative displacement between floors, analyzed as the difference between the average lateral displacements at the top and bottom of the floor considered;

V_{tot} – represents the total seismic shear force on the floor under analysis;

h– represents the height between floors.

For situations where the sensitivity coefficient to relative displacement between floors (θ) is less than or equal to 0.10, second-order effects do not need to be considered. If the value of the coefficient θ is between 0.10 and 0.20, the seismic forces should be increased by a factor equal to 1/(1-θ), and in no case should the value of the coefficient q be greater than 0.30.

3. Results and discussions

3.1. Linear Analysis Results

The tables of the mass values considered in the definition of the dynamic characteristics of the structure, such as the periods, frequencies, modal participation values and the due percentage of mass contribution for each mode are presented in Table 13.

Regarding the first three vibration modes as illustrated in Fig. 18 and Fig. 20, it can be seen that the 1st Mode has the largest modal share (as shown in Fig. 18), which is related to torsion at 86.06%, while the 2nd mode presented in Fig. 19 has a translation in the X direction at 96.83% and the 3rd Mode as illustrated in Fig. 20 with a translation in the Y direction at 98.74%. The periods of 1st, 2nd and 3rd modes of vibration are 1.10, 1.05 and 0.81, respectively. It is worth remembering that these data are calculated from the response spectra proposed by Eurocode 8 combined with the ground accelerations proposed by the World Health Organization for Mozambique. With this, it was possible through the Robot to calculate the frequencies, periods and modal participation values of the structure, thus allowing us to understand that this structure is more influenced by the type II earthquake, as it presents high frequencies.

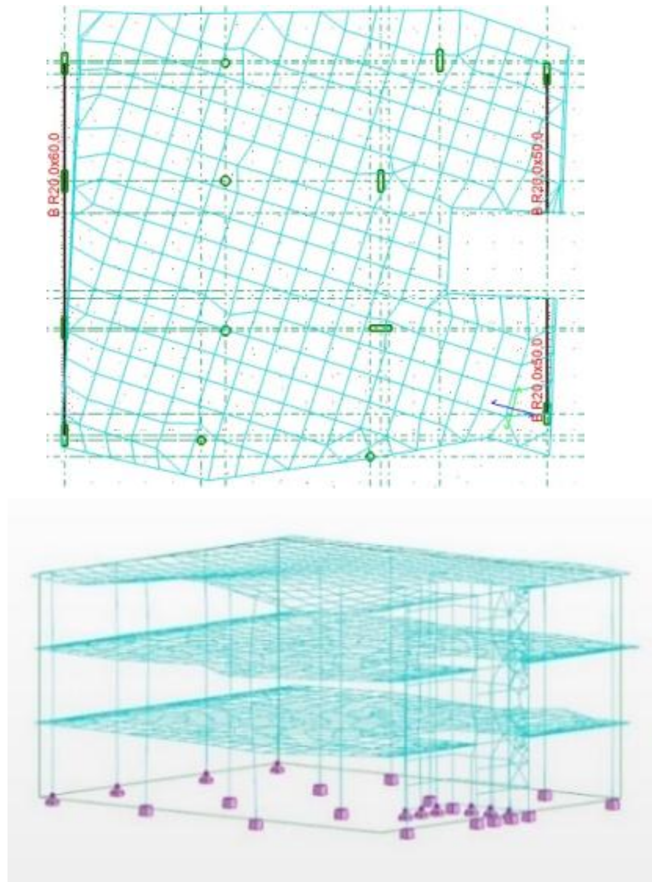

Fig 18. 1st vibration mode of the structure (Source: adapted from Robot by the Author,2024)

3.2. Maximum Displacements of the Structure

The results of the modal analysis below show that the maximum displacements are between 1.7 and 1.9 centimetres as indicated in Table 14. It is worth noting that the maximum displacements are calculated in each direction.

Table 14. Values of maximum displacement of the structure under study (Source: adapted from Robot by the Author, 2024)

	Maximum displacements (cm)					
	X Direction			Y Direction		
Case	q	U_{proj}	U_{Fin}	q	U_{proj}	U_{Fin}
1	3.0	0.10	0.30	3.0	0.10	0.30
2	3.0	0.33	1.00	3.0	0.33	1.00
3	3.0	0.40	1.20	3.0	0.40	1.20
4	3.0	0.13	0.40	3.0	0.13	0.40
5	3.0	0.40	1.20	3.0	0.57	1.70
6	3.0	0.23	0.70	3.0	0.33	1.00
7	3.0	0.43	1.30	3.0	0.63	1.90

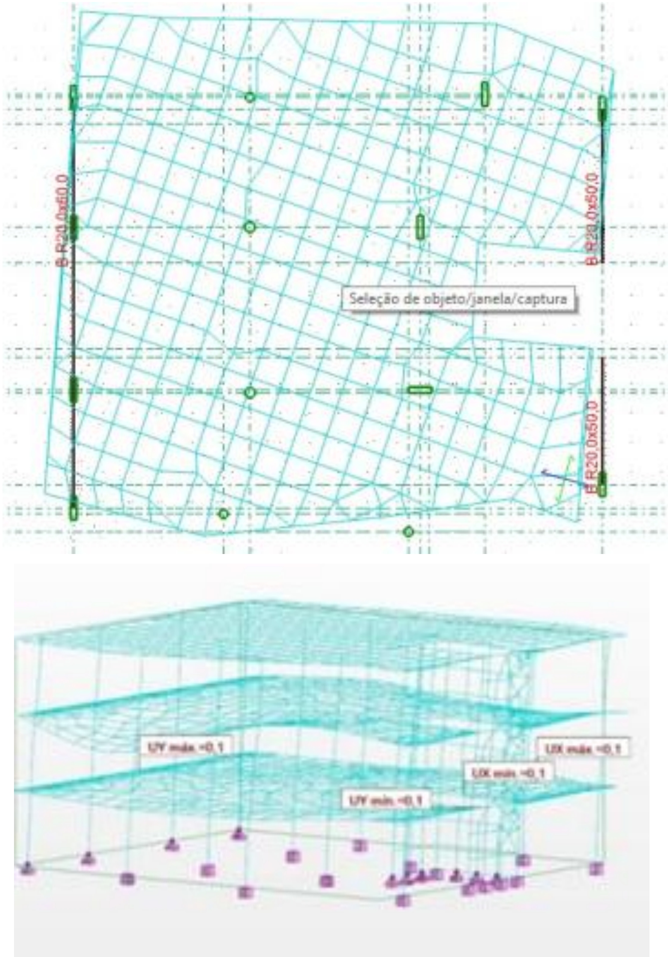

Fig 19. 2nd vibration mode of the structure (Source: adapted from Robot by the Author, 2024)

3.3. Basal Shear of the Structure

To assess the basal cut values, the seismic combinations were analyzed in each direction, to verify the basal cut values by spectral acceleration in all directions. It should be noted that the period used to calculate the shear force at the base was 0.81s, which corresponds to the 3rd vibration mode of the structure, since this of the three modes considered presents the most unfavorable frequency, thus constituting the perfect simulation for calculating the basal shear force as presented in Table 15.

Table 15. Basal shear force values (Source: adapted from Robot by the Author, 2024)

Shear base	Basal shear force (kN)	Seismic coefficient (β)
X Direction earthquake	1,423.00	0.127
Y Direction earthquake	1,282.00	0.115

3.4. Relative Displacements between Floors (Drift) of the Structure

Regarding the relative displacements between floors of the structure in the X and Y directions, these were calculated to comply with the criteria prescribed in regulation EC8, for the control of the limitation of the damage presented in Table 16. It should be noted that this was calculated considering the earthquake with the shortest return period, that is, the conditioning earthquake.

Table 16. Drift between floors of the structure (Source: adapted from Robot by the Author, 2024)

	Relative displacements (cm)					
	X Direction			Y Direction		
Floor	ν	h	d_r	ν	h	d_r
Ground floor	0.55	3.05	0.28	0.55	3.05	0.28
1st Floor	0.55	6.10	0.55	0.55	6.10	0.55
2nd Floor	0.55	9.15	0.83	0.55	9.15	0.83

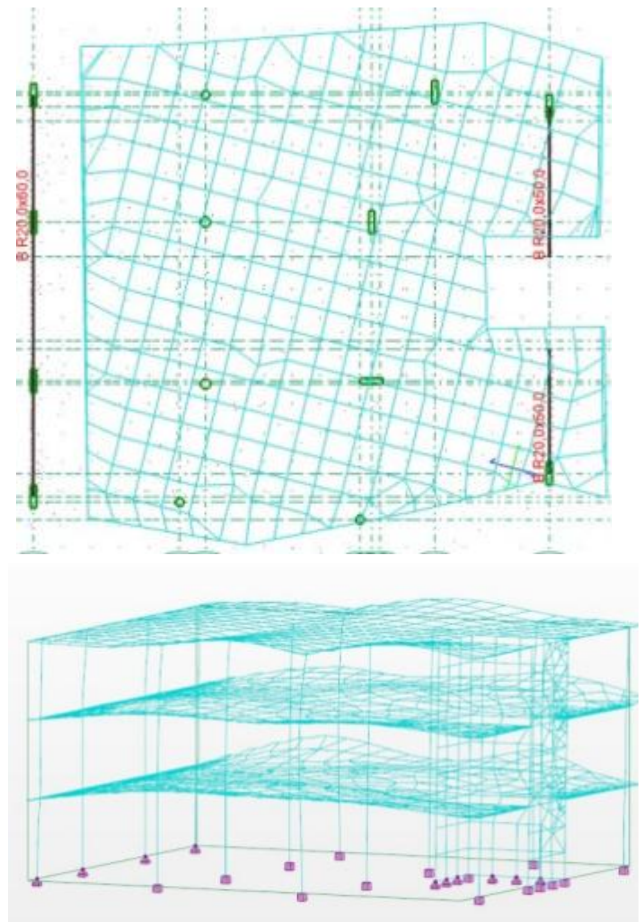

Fig 20. 3rd vibration mode of the structure (Source: adapted from Robot by the Author, 2024)

3.5. Second Order Effects

There is no need to check for second-order effects in the structure since the sensitivity coefficient to relative displacement between floors (θ) is less than or equal to 0.10.

3.6. Earthquake Conditioning

The structure under study is rigid and has a small number of floors and, as verified, the structure has high frequencies and low periods, which causes the structure to vibrate faster. Because of this, the structure tends to be more conditioned by type II earthquakes, also known as near-earthquakes, since they predominantly have higher frequencies.

Table 13. Frequencies, Modal Periods and Modal Participation Factors of the Structure under Study

Case/Mode	Frequency (Hz)	Period(s)	RUX (%)	RUY (%)	CUX (%)	CUY (%)	Total mass Ux (t)	Total mass Uy (t)
40/1	0.91	1.10	86.06	5.50	86.06	5.50	1117.28	1117.28
40/2	0.95	1.05	96.83	75.63	10.77	70.13	1117.28	1117.28
40/3	1.23	0.81	98.27	98.74	1.44	23.12	1117.28	1117.28
40/4	4.25	0.24	99.37	98.95	1.10	0.20	1117.28	1117.28
40/5	4.55	0.22	99.81	99.74	0.44	0.79	1117.28	1117.28
40/6	6.26	0.16	99.85	99.92	0.03	0.18	1117.28	1117.28

Where:
CUX – represents the modal participation of the structure's rotation in the x direction;
CUY – represents the modal participation of the structure's rotation in the y direction.
RUX – represents the modal participation of translation of the structure in the x direction;
RUY – represents the modal participation of translation of the structure in the y direction

Therefore, when the fundamental frequency of the building tends to equal the frequency of the earthquake, a phenomenon called resonance occurs, where the vibrations amplify the effect of the earthquake on the structure.

3.7. Evaluation of Results

After performing the structural seismic analysis of the building under study, it was found that it meets the EC8 criteria, regarding the minimum number of modes considered in the analysis ($k = 6 \geq k_{min} = 6$)and the vibration period of the last mode, which is less than $(T_6 = 0.16s \leq 0.20s)$, therefore the modal analysis can be considered valid, and the modes considered present a significant contribution since the sum of the effective modal masses is greater than 90% of the global mass of the structure and all modal masses are greater than 5% of the structure.

Furthermore, this structural system can be defined as non-resistant, that is, not stable, although it is not subject to drastic translational deformations in x and y, and torsion. As stated, the first three modes were crucially evaluated because they are decisive in characterizing the deformation mode of the structure under study. After this analysis, it was found that the structure does not respond satisfactorily to the displacements imposed by the action of earthquakes. Due to the torsion effect observed in the first vibration mode, the first two vibration modes must be characterized by a translational movement, thus compromising the structural seismic resistance of the building under study.

The translation in x observed in the 2nd vibration mode is also, in a certain way, crucial to the seismic response of the building, due to the unfavourable arrangement of the walls, since 90% of them are arranged in the same direction, which means that the structure is favourable to the translational movements in Y observed in the third vibration mode. Faced with this dilemma, in this case, the translation in X, an in-situ solution is occasionally found, which is the existence of neighbouring infrastructures protecting the structure against seismic action in this direction.

4. Conclusions

- The primary purpose of this study was to conduct a comprehensive seismic analysis of a three-story reinforced concrete building to assess its vulnerability and structural integrity under earthquake conditions, ensuring safety and resilience in seismically active regions.
- The building displayed high frequencies and low periods across vibration modes, indicating a tendency for rapid vibration under seismic action. It was found to be more influenced by Type II (high-frequency) earthquakes, which led to torsional displacement in the 1st mode (86.06%) and translational displacements in the X (96.83%) and Y directions (98.74%) for the 2nd and 3rd modes, respectively.
- While the building met basic EC8 criteria for seismic analysis (i.e., six vibration modes considered and a final mode period of 0.16s), several structural vulnerabilities were observed. Wall orientations contributed to heightened translational movements, especially in the X direction (90% of walls oriented along X). Torsion was dominant in the 1st mode, indicating a need for adjustments to improve seismic resilience in similar structures.
- The Maximum displacements were between 1.7 cm and 1.9 cm in the X and Y directions, underscoring areas of concern under seismic forces. The basal shear force reached 1,423 kN in the X direction and 1,282 kN in the Y direction, with a seismic coefficient of 0.127 and 0.115, respectively, which aligns with the most critical seismic mode at a period of 0.81s.

Recommendations and Future Prospects:

- Given Mozambique's seismic risk areas, especially in Maputo, Beira, and Rift Valley provinces (Niassa, Sofala, Manica, Tete, Inhambane, and Gaza), there is a dire need for Localized Seismic Standards to allow structures to better withstand regional seismic events based on accurate local data. Development of national standards could address specific site conditions, while parametric studies would help correlate local seismic characteristics with those in other standards like Eurocode 8.
- For new buildings in seismically active regions, recommendations include a diverse wall arrangement to distribute seismic forces evenly, incorporation of cross-walls to enhance stiffness and stability, use of advanced modeling techniques and nonlinear analysis for detailed structural behavior assessment, implementation of deep foundations and consideration of soil-structure interaction, and regular structural inspections and seismic retrofitting as necessary. These measures aim to improve the seismic resilience of new constructions and ensure their safety and integrity.

- Establishing a national seismic observatory could provide Mozambique with independent seismic data collection, promote research into localized seismic design approaches and minimize losses.
- Future research should consider the assessment of structural responses across different building geometries and soil types to establish best practices. Furthermore, the evaluation of the effectiveness of traditional construction practices concerning seismic resistance in Mozambique, as well as the investigation of the relationship between disorderly urban development and seismic vulnerability should also be explored.

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Conflict of Interest Statement

The authors declare that there is no conflict of interest in the study.

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