ASSESSMENT OF SEISMIC HAZARD AND STRUCTURAL SAFETY OF BUILDINGS IN BEIRA AND CHIMOIO

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ABSTRACT :

The seismic hazard and structural safety in Beira and Chimoio (Mozambique) are evaluated, since these cities were identified as being relatively close to potential seismic faults. Several parametric variations were made, in order to take into account the unknowns related to the size of the seismic fault, and the distance between the fault and the city. With the seismic scenarios defined, and through the application of different attenuation equations, the parameters which characterize the seismic motions were obtained for each scenario considered. With the obtained data, the seismic performance of selected buildings in the defined locations was assessed through the application of a recently developed displacement based method (DBELA).

KEYWORDS: Seismic hazard, structural safety, displacement based earthquake loss assessment.

1. INTRODUCTION TO THE SEISMICITY OF MOZAMBIQUE

Most of the regional seismic activity in Southern Africa is connected to the East Africa rift. The rift's activity is characterized by shallow earthquakes (focus depths of no more than 35 km) and by normal or strike-slip faults [1]. Mozambique is part of the East African rift extending south from the Gulf of Aden for more than 3000 km. Though not for long considered completely an earthquake free country, in Mozambique the preoccupation with seismic hazard was very small – if not at all inexistent – until the seismic events that took place in February 2006. These events, which peaked with the 7,0 magnitude Machaze earthquake of the 23rd February 2006, showed that the possibility of seismic occurrence in the country is real and should not be lightly put aside. According to the United States Geological Survey in the last 30 years around 190 seismic events took place in Mozambique all with focus depths of less than 35 km; also the largest earthquake to take place in the rift within the last 100 years might have had a magnitude of 7,6 and more than 50% of the registered events had magnitudes above 4,0 and 16 had magnitudes above 5,0 [2]. Since February 2006 there has been an increase in the seismic activity in the region of Machaze, which is located about 500 km north of Maputo and about 200 km from Beira and Chimoio. This activity was of such intensity that almost 40% of the seismic events registered in the country since 1973 (Figure 1 – left) took place throughout 2006 (Sousa [1]).

2. SEISMIC HAZARD ASSESSMENT

Seismic hazard can be defined as the possibility of occurrence of potentially destructive effects caused by an earthquake in a specific place and at a given time period [1]. A seismic hazard assessment has the objective of determining the nature and intensity of predicted future seismic motions at a specific location, and it might include the assessment of the probability of occurrence of these motions.
One of the fundamental choices that must be made when carrying out a seismic hazard assessment is whether to follow a deterministic or probabilistic approach. Independently of the chosen approach there are two basic elements to any hazard assessment: (i) A seismicity model, which allows to define the location and magnitude of future seismic events – in other words it establishes seismic occurrence scenarios; (ii) A model that allows to estimate the characteristic parameters of a given seismic motion, corresponding to the defined seismic scenarios – this model is usually an attenuation equation.

Due to the constraints found in the base study of this paper (inexistence of previous studies and very limited information), it was decided to use a series of deterministic analysis. A main seismicity model was defined, and several parametric variations were made to the model to account for a reasonable array of seismic scenarios (covering eventual uncertainties related to size and location of the seismic fault). With the scenarios defined, a series of attenuation equations were applied to determine the characteristic parameters of the seismic motions.

2.1. Seismicity Models considered

Beira is a city with about 400 000 inhabitants distributed over a 633 km2 area. The map shown in Figure 1 (middle) identifies Fault "A" as the closest seismic source to the city, at about 25 km and extending over a distance of around 115 km. Chimoio is a city with about 170 000 inhabitants distributed over a 174 km2 area. The map shown in Figure 1 (right) identifies Fault "B" as the closest seismic source to the city, at about 80 km and extending over a distance of around 150 km. For the city of Beira, the scenarios considered correspond to the following parametric variations: distance to source (r) of 15, 25 and 35 km and fault size 65 and 115 km. For the city of Chimoio, the scenarios considered correspond to the following parametric variations: distance to source (r) of 70, 80 and 90 km and fault size 100, 125 and 150 km [1].

Figure 1 – Seismic activity (1973-2006) and Location of faults A (Beira) and B (Chimoio) in central Mozambique

Using Eqn 2.1 of Wells and Coppersmith [3] the earthquake magnitudes shown in Table 2.1 were obtained.

$$M_W = 4,38 + 1,49 \times log(L) \tag{2.1}$$

Table 2.1 Magnitudes corresponding to sizes of Faults *"A"* and *"B"*

	A (Beira)		*B (Chimoio)*		
L [km]	65	115	100	125	150
M_W	7,1	7,5	7,4	7,5	7,6

2.2. Attenuation Equations considered

The inexistence of attenuation equations developed for this specific region, led to the use of attenuation equations developed for other locations judged to have similar characteristics [1, 4, 5]. The already mentioned base study [1] identified 2 attenuation equations as the ones to consider for further research and use in this work: expressions of Akkar and Bommer [6] and expressions of Campbell and Bozorgnia [7], with the main limitations presented in Table 2.2. The objective of this analysis was to generate displacement spectra that could be used to carry out the structural safety assessment, which was the main objective of the study [1, 6, 7].

Table 2.2 Limitations of the chosen attenuation equations

	M_w	T [s]
Akkar and Bommer	5 – 7,6	0,05 – 4,00
Campbell and Bozorgnia	4 – 7,5	0,01 – 10,00

2.3. Results obtained for the Attenuation Equations

The most relevant results, obtained through the use of the Akkar and Bommer [6] and Campbell and Bozorgnia [7] expressions, are presented as displacement spectra (displacements – Sd – in cm; and period – T – in seconds) in Figures 1 and 2. A brief comparison of the results obtained through the application of the different equations, made for the fundamental periods of the buildings analyzed in Beira and Chimoio, was already presented graphically [1] for periods T = 0,3; 0,6 and 1,0 s. The results obtained by the application of these expressions are very similar, since the curves have a nearly coincident development. It is also noteworthy that the evolution of the curves is the same, independently of the magnitudes considered [1, 4, 5]. Table 2.3 gives the spectral displacements obtained through the application of both methodologies, for Beira and Chimoio.
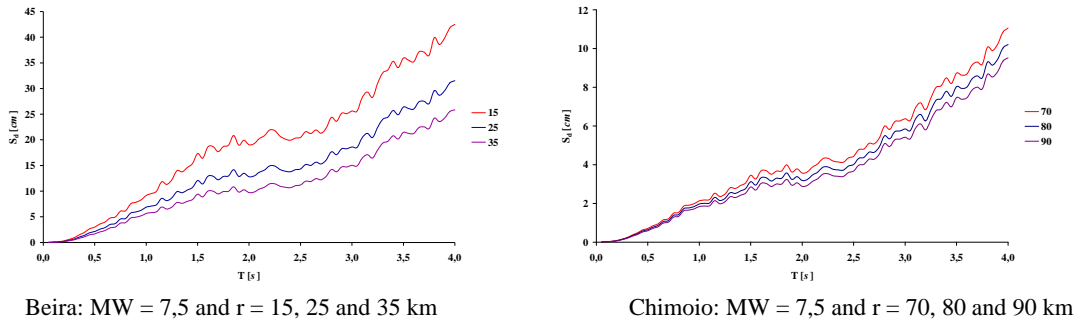

Beira: MW = 7,5 and r = 15, 25 and 35 km   Chimoio: MW = 7,5 and r = 70, 80 and 90 km

Figure 2 – Spectral displacements (Akkar and Bommer) for Beira and Chimoio

Beira: MW = 7,5 and r = 15, 25 and 35 km   Chimoio: MW = 7,5 and r = 70, 80 and 90 km
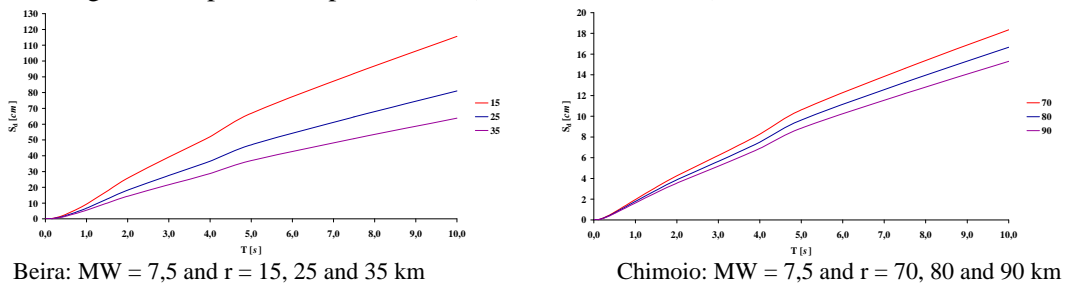
Figure 3 – Spectral displacements (Campbell and Bozorgnia) for Beira and Chimoio

Table 2.3 Spectral displacements S_d (cm) for Beira Chimoio (A&B Akkar-Bommer;C&B Campbell-Bozorgnia)

Beira		M_W	7,1	7,5		*Chimoio*		M_W	7,4	7,5	7,6
T [s]	**r [km]**	V_{30} [m/s]	\multicolumn{2}{c}{200}		**T [s]**	**r [km]**	V_{30} [m/s]	\multicolumn{3}{c}{500}	
0,3	15	A & B	0,86	0,94		**0,3**	70	A & B	0,23	0,24	0,25
		C & B	1,02	1,06				C & B	0,31	0,32	-
	25	A & B	0,58	0,66			80	A & B	0,21	0,22	0,23
		C & B	0,73	0,78				C & B	0,28	0,29	-
	35	A & B	0,44	0,52			90	A & B	0,19	0,20	0,21
		C & B	0,59	0,64				C & B	0,25	0,26	-
0,6	15	A & B	3,56	3,99		**0,6**	70	A & B	0,90	0,95	1,00
		C & B	3,61	4,12				C & B	0,93	1,00	-
	25	A & B	2,47	2,90			80	A & B	0,82	0,87	0,91
		C & B	2,56	3,01				C & B	0,84	0,90	-
	35	A & B	1,92	2,33			90	A & B	0,75	0,80	0,85
		C & B	2,04	2,45				C & B	0,77	0,83	-
1,0	15	A & B	7,75	9,16		**1,0**	70	A & B	1,98	2,13	2,28
		C & B	7,57	9,41				C & B	1,81	1,96	-
	25	A & B	5,52	6,87			80	A & B	1,83	1,97	2,11
		C & B	5,32	6,82				C & B	1,64	1,78	-
	35	A & B	4,39	5,65			90	A & B	1,70	1,84	1,97
		C & B	4,20	5,51				C & B	1,50	1,64	-

3. STRUCTURAL SAFETY ASSESSMENT

In this sections it is briefly described the methodology in which was based the assessment of the structural performance of the selected buildings (DBELA – Displacement Based Earthquake Loss Assessment). For a more detailed description of the DBELA methodology please refer to Crowley et al. [8]. This procedure considers 6 distinct performance states (3 structural states and 3 non-structural states). The expressions developed and detailed in [8] consider material and geometrical characteristics, relating the height of the building to a period corresponding to each of the 6 limit states – allowing for a direct comparison between the displacement capacity of the building (for a given state) and the displacement imposed on the building by the seismic motion (calculated in the previous section of the study). The utilization of this method also implies the definition of the expected collapse mechanism of the structure, this can be done taking into account the type of construction, year of construction, evidences of a resistant/frail first storey, etc. In this case it was assumed that the failure mechanism would be of the column-sway (soft storey) type [1].

3.1. Structural Displacement Capacity

The equations used herein describe the displacement capacity of an equivalent single degree of freedom structure, and give the displacement capacity in the centre of the resultant of seismic forces of the original structure. For the type of structure being analyzed the displacement is obtained by multiplying the base rotation of the structure by an effective height. The effective height is obtained by multiplying the total height by an effective height coefficient (ef_h) which is equal to 0,67 in the pre-yield stage.
In the post-yield stage this coefficient is obtained using Eqn 3.1 developed and detailed in [9] and [10].

$$ef_h = 0,67 - 0,17 \frac{\mu_{Lsi} - 1}{\mu_{Lsi}} \tag{3.1}$$

However the ductility can also be determined once the yield displacement has been determined, which leads to an iterative process. Crowley et al. [8] suggest the use of an initial value of $ef_h = 0,60$ in Eqn 3.2 (deduced in [10]), to estimate the ductility which should be then introduced in Eqn 3.1 to get a better estimate of ef_h (only one iteration is necessary)

$$\mu_{Lsi} = 1 + \frac{\left(\varepsilon_{C(Lsi)} + \varepsilon_{S(Lsi)} - 2,14\varepsilon_y \right) h_c}{0,86 ef_h H_T \varepsilon_y} \tag{3.2}$$

where $\varepsilon_{C(Lsi)}$ and $\varepsilon_{S(Lsi)}$ are the maximum admissible extensions for steel and concrete for limit state i; ε_y is the yield extension of steel; h_c is the height of the pillar's section and H_T is the total height of the original structure. The yield displacement capacity is determined by Eqn 3.3 [11] where Δ_{Sy} is the structural yield displacement capacity (limit state 1) and h_s is the height of the first storey. The post-yield structural displacement capacity (Δ_{SLsi}) is obtained through the application of a plastic displacement component (Eqn 3.4).

$$\Delta_{Sy} = 0,43 ef_h H_T \varepsilon_y \frac{h_s}{h_c} \tag{3.3}$$

$$\Delta_{SLsi} = 0,43 ef_h H_T \varepsilon_y \frac{h_s}{h_c} + 0,5 \left(\varepsilon_{C(Lsi)} + \varepsilon_{S(Lsi)} - 2,14\varepsilon_y \right) h_s \tag{3.4}$$

3.2. Non-structural displacement capacity

The non-structural analysis of column-sway structures assumes the concentration of damage in the base floor of the building. The determination of the non-structural displacement capacity starts by comparing the displacement corresponding to the limit state in analysis (Δ_{NSi1st}) with the yield displacement of the base floor (Δ_{Sy1st}), as defined by Eqns 3.5 and 3.6 respectively, where v_i is the drift capacity corresponding to limit state i.

$$\Delta_{NSi1st} = v_i h_s \qquad (3.5)$$

$$\Delta_{Sy1st} = 0,43 \, ef_h \, h_s \, \varepsilon_y \, \frac{h_s}{h_c} \qquad (3.6)$$

If Δ_{NSi1st} is smaller than Δ_{Sy1st} the structure is in its pre-yield state, and Eqn 3.7 should be used

$$\Delta_{NSLsi} = 0,67 \, v_i \, H_T \qquad (3.7)$$

where Δ_{NSLsi} is the non-structural displacement capacity corresponding to limit state i.
However, if Δ_{NSi1st} is larger than Δ_{Sy1st}, the structure is in its post-yield stage and Eqn 3.8 should be used.

$$\Delta_{NSLsi} = v_i \, h_s + 0,43 \left(ef_h \, H_T - h_s \right) \varepsilon_y \, \frac{h_s}{h_c} \qquad (3.8)$$

3.3. Period-Height relation

Crowley et al. [8] suggested simple formulas to relate the height of reinforced concrete frames to their period. Eqn 3.9 relates the total height of the building to its yield period (T_y). Eqn 3.10 relates the total height of the building to the periods corresponding to different post-yield limit states, where T_{Lsi} is the post-yield period (corresponding to limit state i).

$$T_y = 0,1 H_T \qquad (3.9)$$

$$T = T \sqrt{\mu} \qquad (3.10)$$

3.4. Displacement demand

As it has already been mentioned this methodology uses displacement spectra to represent the demand imposed by seismic motions. To account for hysteretic energy dissipation the critical damping factor of Eqn 3.11 [8] is used, where a and b should be considered as $a = 25$ and $b = 0,5$ [12]. Most displacement spectra are obtained for the reference value of critical damping factors $\xi = 5\%$. When different from the reference value, the damping factor obtained by Eqn 3.11 can be used to obtain a reduction coefficient [13] which should be used to convert the response spectra corresponding to the value of $\xi = 5\%$.

$$\xi = a \left(1 - \frac{1}{\mu} \right) + 5 \qquad (3.11)$$

3.5. Case studies

Here are described the buildings analysed in the base study of this paper [1]. The buildings are identified as Building A, Building B and Building C. For more detailed information about these case studies please refer to Sousa [1] and also to Sousa, Barros and Bommer [14].
Building A is a 14 storey hotel built in the 1960s. The building "data" considered in the study was: $h_s = 3,90 \, m$; $h_c = 0,59 \, m$; $H_T = 44,20 \, m$. Extensions/drifts for the different limit states: $\varepsilon_y = 0,002$; $\varepsilon_{S(Ls2)} = 0,013$ and $\varepsilon_{S(Ls3)} = 0,018$; $\varepsilon_{C(Ls2)} = 0,005$ and $\varepsilon_{C(Ls3)} = 0,008$; $v_1 = 0,002$; $v_2 = 0,004$ and $v_3 = 0,008$.
Building B is a 3 storey religious social centre and residence built in the 1960s. The building "data" considered in the study was: $h_s = 4,10 \, m$; $h_c = 0,24 \, m$; $H_T = 10,10 \, m$. Extensions/drifts for the different limit states: $\varepsilon_y = 0,002$; $\varepsilon_{S(Ls2)} = 0,013$ and $\varepsilon_{S(Ls3)} = 0,018$; $\varepsilon_{C(Ls2)} = 0,005$ and $\varepsilon_{C(Ls3)} = 0,008$; $v_1 = 0,002$; $v_2 = 0,004$ and $v_3 = 0,008$.

Building C is a Hospital in construction at the time of the study, with 2 blocks (C_1 and C_2). C_1 building "data" considered in the study was: $hs = 3,15\ m$; $hc = 0,24\ m$; $HT = 3,15\ m$ and C_2 was: $h_s = 3,35\ m$; $h_c = 0,32\ m$; $H_T = 6,50\ m$. Extensions/drifts for the different limit states: $\varepsilon_y = 0,002$; $\varepsilon_{S(Ls2)} = 0,010$ and $\varepsilon_{S(Ls3)} = 0,040$; $\varepsilon_{C(Ls2)} = 0,004$ and $\varepsilon_{C(Ls3)} = 0,010$; $v_1 = 0,001$; $v_2 = 0,003$ and $v_3 = 0,005$. Because it's a newer building better concrete confinement was considered, which would allow for bigger limit extensions when compared to the other cases. However, being a Hospital, lower extensions and drifts were considered.

3.6. Results for the buildings location in Beira and Chimoio

The following tables – dependent on the displacement capacities and associated periods for the different limit states – briefly present the results of the structural safety assessment carried out for the three different buildings, based upon the corresponding performance analysis of each building, located either in Beira or in Chimoio. Because of lack of space the results of building C are not presented herein and can be consulted in [1, 14].

3.6.1 Building A

Table 3.1 Displacement capacities and periods – *Building A*

	Structural Limit State			Non-structural Limit State		
	1	**2**	**3**	**1**	**2**	**3**
$\Delta_{S/NS}$ *[m]*	0,168	0,187	0,200	0,059	0,118	0,174
T [s]	4,42	4,75	4,95	4,42	4,42	4,51

Table 3.2 *Building A* in Beira and Chimoio: (C&B – *Campbell and Bozorgnia*; SLS – Structural Limit States; NSLS – Non-structural Limit States; 👍 – does not exceed the limit; × – exceeds the limit)

Beira		$M_W = 7,1$						$M_W = 7,5$					
		SLS			NSLS			SLS			NSLS		
	r (km)	1	2	3	1	2	3	1	2	3	1	2	3
C&B	15	×	×	×	×	×	×	×	×	×	×	×	×
	25	×	×	×	×	×	×	×	×	×	×	×	×
	35	×	×	×	×	×	×	×	×	×	×	×	×

Chimoio		$M_W = 7,4$						$M_W = 7,5$					
		SLS			NSLS			SLS			NSLS		
	r (km)	1	2	3	1	2	3	1	2	3	1	2	3
C&B	70	👍	👍	👍	×	👍	👍	👍	👍	👍	×	👍	👍
	80	👍	👍	👍	×	👍	👍	👍	👍	👍	×	👍	👍
	90	👍	👍	👍	×	👍	👍	👍	👍	👍	×	👍	👍

3.6.2 Building B

Table 3.3 Displacement capacities and periods – *Building B*

	Structural Limit State			Non-structural Limit State		
	1	**2**	**3**	**1**	**2**	**3**
$\Delta_{S/NS}$ *[m]*	0,099	0,120	0,134	0,014	0,027	0,051
T [s]	1,01	1,14	1,22	1,01	1,01	1,01

Table 3.4 *Building B* in Beira and Chimoio: (A&B – *Akkar and Bommer*; C&B – *Campbell and Bozorgnia*; SLS –Structural Limit States; NSLS–Non-SLS; ✓ –does not exceed the limit; × –exceeds the limit)

Beira		$M_W = 7{,}1$						$M_W = 7{,}5$					
		SLS			NSLS			SLS			NSLS		
	r (km)	1	2	3	1	2	3	1	2	3	1	2	3
A&B	15	✓	✓	✓	×	×	×	✓	✓	✓	×	×	×
	25	✓	✓	✓	×	×	✓	✓	✓	✓	×	×	×
	35	✓	✓	✓	×	✓	✓	✓	✓	✓	×	×	×
C&B	15	✓	✓	✓	×	×	×	✓	✓	✓	×	×	×
	25	✓	✓	✓	×	×	×	✓	✓	✓	×	×	×
	35	✓	✓	✓	×	×	✓	✓	✓	✓	×	×	×

Chimoio		$M_W = 7{,}4$						$M_W = 7{,}5$						$M_W = 7{,}6$					
		SLS			NSLS			SLS			NSLS			SLS			NSLS		
	r (km)	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
A&B	70	✓	✓	✓	×	✓	✓	✓	✓	✓	×	✓	✓	✓	✓	✓	×	✓	✓
	80	✓	✓	✓	×	✓	✓	✓	✓	✓	×	✓	✓	✓	✓	✓	×	✓	✓
	90	✓	✓	✓	×	✓	✓	✓	✓	✓	×	✓	✓	✓	✓	✓	×	✓	✓
C&B	70	✓	✓	✓	×	✓	✓	✓	✓	✓	×	✓	✓	-	-	-	-	-	-
	80	✓	✓	✓	×	✓	✓	✓	✓	✓	×	✓	✓	-	-	-	-	-	-
	90	✓	✓	✓	×	✓	✓	✓	✓	✓	×	✓	✓	-	-	-	-	-	-

3.7. Analysis of results

Even though it must be mentioned that the analyses carried out assess the specific behavior of the buildings that were considered, it is possible to make some broad conclusions that might allow predicting the behavior of similar buildings in the same location under similar circumstances. To this end it is considered that: Building A behavior is representative of that of tall buildings (e.g. between 8 and 14 storeys), Building B behavior is representative of medium buildings (e.g. 3 to 5 storeys) and Building C behavior is representative of low buildings (e.g. 1 to 2 storeys). This approximation is however an assumption, for even though the selection of the building was made with the objective of selecting those which would represent a certain class, there is no unequivocal information that allows us to confirm this selection [1, 14].

As such, even though their performance can be used as a guide for similar buildings in the same location, one must always bear in mind that the results presented here pertain exclusively to the analyzed buildings.

The analysis of these buildings in Beira raises preoccupying results. For Building A it is predicted that it will exceed all structural limit states for all the evaluated scenarios – therefore there is a risk of collapse. The shorter buildings do not appear to be subject to potential structural damage – except for Building C_2 which can expect to suffer moderate structural damage for the scenarios in which the seismic source is at 15 km. Considering that Building C is a hospital, such structural damage can be considered as indicative of poor performance. In what concerns non-structural damage, the analysis carried out predicts extensive non-structural damage for all buildings and for all the scenarios that were considered. Given the apparent risk to Beira it is recommended that the seismic source considered in this study be exhaustively studied. The information about this fault should be expanded through record analysis as well as field work. This would allow a better assessment of the danger posed by the fault, and a better study of the consequences and preventive measures to consider [1, 14]. In view of the poor performance of the tall building class, it is considered important that the study of Sousa [1] be expanded to other tall buildings to assess if their performance is as poor as predicted.

The analyses of these buildings in Chimoio show that, under the used scenarios, none of the building types analyzed in this study are in danger of suffering structural damage in this city. The main reason for this good performance is the considerable distance between the seismic source and the city (about 80 km), and to a lesser extent the considerable bearing capacity of the soils in this region.

On the non-structural front the results show that most building types will suffer moderate non-structural damage in the case of a seismic event. Building C_2 is an exception because, accounting for the lower drift limits allowed for this building (considering that it is a Hospital building), it suffers complete non-structural damage, which is obviously unacceptable for such a vital structure – especially in the aftermath of an earthquake [1, 14].

4. CONCLUSIONS

One of the limitations of this structural safety assessment methodology is that it works as an indicator, that is: it allows researchers and developers to identify structures which when subjected to certain seismic motions will exceed specific structural/non-structural limit states. However it does not allow specific conclusions on the weaknesses of each structure or on the structural elements that should be reinforced, preventing structural failure. Thus, complementing the use of this methodology, a more sophisticated structural analysis should be carried out for buildings identified as having poor performance in order to determine which structural elements could be reinforced, in order to improve the performance of the structure. A point to be considered in future analysis of buildings in Beira, is the existence of high water table in the city foundation soils. It is expected that liquefaction of these soils will occur under moderate to severe seismic motions. The consideration of this factor will surely result in an increased seismic risk to buildings located in Beira.

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