Should Standard Building Structures in Brazil be Guaranteed by a Seismic Resistant Design?

A. A. D. Almeida
J. L. Roehl
and J. L. Alves
Departamento de Engenharia Civil
Pontifícia Universidade Católica do
Rio de Janeiro, PUC-Rio
Rua Marquês de São Vicente, 225
22453-900 Rio de Janeiro, RJ. Brazil
adiniz@civ.puc-rio.br
roehl@civ.puc-rio.brr

Brazilian territory is situated on an intraplate region with low seismic activity, and so, it is important to answer an old question that remains unanswered: "Should standard building structures in Brazil be guaranteed by a seismic resistant design?" In this way , a methodology is described and used to evaluate the annual failure probability of a structure model subjected to ground motions compatible with a defined provincial seismicity; the model strength is taken as the maximum shear and overturning moment at the base. One considers five different structure models for each Brazilian state capital city and evaluates the annual failure probability which provides arguments to answer the above question.
***Keywords**: Seismic risk probability analysis, seismic demand, annual failure probability*

Introduction

The above title is an old question which remains unanswered.

A trivial and frequent return is negative and is justified by the absence of specific recommendations in Brazilian structural design codes to cover this subject and one may try to close definitely the discussion with the statement: "After all, there are no earthquakes in Brazil!". However, this argument is false and the question remains superficially and insufficiently answered.[1]

Since the late sixties, high sensibility seismographic stations have been installed in the Brazilian territory and they have recorded inside Brazilian borders and in neighboring region frontiers tectonic ground motions of relatively low magnitude, in the Richter scale, but with MM intensity grade sufficiently high to affect civil structural systems (Berrocal 2001). These instrumentally recorded evidences, together with the also reported macro seismic information, justified the statement: "Yes, there are earthquakes in Brazil, and they may affect structural systems".

On the other side, geophysical, geological and seismological studies (Berrocal 2001) identify that the Brazilian territory is situated on an intraplate region of the Earth crust, with most of the subsoil formations older than quaternary and, in consequence, a low tectonic activity, approaching a homogeneous seismicity model, which can be admitted within roughly defined seismic provinces.

This report is an initiative to clarify the subject. In this way, recognizing that the seismic manifestation is random, the probabilistic risk formulation suggested by Kramer (1996) and Melchers (1987) is used to assess the seismic hazard and the probability of undesirable effects on structures. The demand uncertainties are expressed by the cumulative distribution of annual probabilities of seismic global action levels on structures. On the offer side, it is used the cumulative distribution probability of the structure capacity to withstand the seismic excitations, independently of the structure lifetime.

The following strategy is used for the analysis development:

- the Brazilian territory division into seismotectonic provinces, as proposed by Almeida (2002), is enhanced and the state capital cities are selected as "sites";
- for these "sites", seismic hazard curves are developed according to USNRC recommendations (1997), referred to the maximum ground acceleration;
- a standard building structure model is assembled by the superposition of a variable number of simple floor frames as a paradigm for state capital city buildings;
- the total shear (Qb) and overturning moment (Mb) actions are used to characterize, in a global manner, the wind and seismic actions on the structure model base;
- it is assumed that the structure model design follows the recommendations of NBR-6123 – Wind Forces on Buildings (ABNT 1988) and their associated wind global actions on the model base are used to quantify a residual structure capacity to support accidental horizontal loads, under a normal distribution;
- then, the cumulative seismic demand is determined, in terms of the global actions at the structure model;
- a fragility curve is determined, representing the conditional probability distribution of the demand to overpass the capacity of the model to support accidental horizontal loads;
- the convolution of the seismic hazard curve with the fragility curve furnishes the annual probabilities of the seismic actions to overpass the strength capacity;
- it is further assumed that wind and seismic are mutually excluding actions and it is used the above results to answer the old question.

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Nomenclature

a_{max} = *a given level of maximum seismic ground acceleration*
b = *a given demand level*
C = *capacity*
D = *demand*
d_k = *column section dimension*
f_0 = *natural frequency*
F_D = *demand accumulative function*
f_C = *capacity probability density function*
f_D = *demand probability density function*
Fi = *horizontal drag force applied on half of building height*
F_{Mb} = *overturning moment accumulative function*
F_{Qb} = *total shear accumulative function*
H_{RR} = *transfer function*
L_T = *probability of a given demand level not to be exceeded during the interval (0, t*)*
M_b = *overturning moment at the building base*
$P_{f/amax}$ = *failure conditional probability, given* a_{max}.
$P_{f\,anua}$ = *annual failure probability*
P_h = *seismic hazard probability*
PSD_{AgAg} = *ground acceleration power spectral density function*
PSD_{RR} = *structure response power spectral density function*
Q_b = *the total shear force at the building base*
t^* = *stationary ground motion duration*
V = *wind basic velocity*
Z = *probabilityto start below a given demand level*

Greek Symbols

λ = *i-th order spectral moment*

Development

Structure Model

Initially, it is assumed a concrete building structure with five floors. Each floor has 270t of mass which is lumped in equal parts on the top of two diagonally opposite columns. Beams and columns are modeled by frame elements with constant sections. As a conservative attitude a 2% viscous damping is assumed. This standard floor model is successively expanded vertically by the addition of other similar systems until a total of 25 floors is achieved, Fig. 1.

According to the model height in meters, the structure models are designated Frame20, 40, 60, 80 and Frame100. For each five floor set, the necessary column section dimension, $d_{k,}$ is calculated according to standard design rules, resulting equal to 1.45, 1.33, 1.16, 0.96 and 0.71m, from bottom to top. Table 1 shows, for each model, the natural frequencies at the first thirty vibration modes. These structure low frequencies are in the main range of ordinary earthquake spectra: 0.1-15 Hz.

Table 1. Natural frequencies, Hz.

Mode	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Frame20	5.0	5.2	5.2	5.4	13.4	13.7	14.3	14.9	18.5	18.8	21.7	21.7	21.8	21.8	29.9
Frame40	2.4	2.6	2.6	3.0	6.9	7.1	7.7	8.8	11.1	12.0	13.3	13.3	13.8	13.9	17.4
Frame60	1.4	1.5	1.6	2.1	4.4	4.6	5.0	6.2	7.8	8.8	9.0	9.3	10.0	10.6	12.6
Frame80	0.9	1.0	1.0	1.6	3.1	3.3	3.5	4.8	5.9	6.5	6.5	6.9	7.8	8.9	9.5
Frame100	0.7	0.7	0.7	1.2	2.2	2.4	2.5	3.6	4.3	4.8	4.9	5.0	6.0	6.9	6.9
Mode	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**	**26**	**27**	**28**	**29**	**30**
Frame20	29.9	29.9	30.0	39.1	39.2	39.2	39.2	48.9	49.0	77.6	77.9	100.7	101.2	115.7	116.2
Frame40	17.5	17.7	17.7	20.9	20.9	20.9	21.0	24.3	24.3	24.3	24.4	26.3	26.4	28.3	28.3
Frame60	12.8	13.3	13.3	15.7	15.8	15.8	16.1	18.1	18.2	18.2	18.4	18.6	18.7	20.4	20.5
Frame80	9.9	10.6	10.6	12.1	12.5	12.5	13.0	14.1	14.5	14.8	14.9	15.2	15.2	16.8	16.8
Frame100	7.4	8.1	8.5	9.1	9.6	9.7	10.2	10.8	11.2	11.5	11.6	12.0	12.1	13.2	13.3

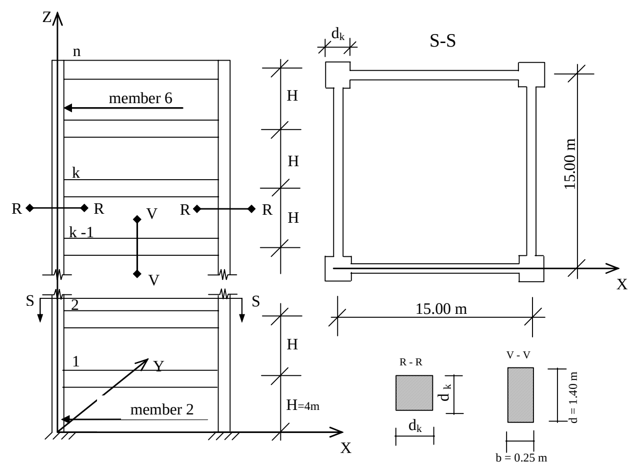

Figure 1. Standard structure model.

Lateral Loads Capacity Distribution

The structure model design is assumed to follow the recommendations of *NBR-6123 –Wind Forces on Buildings* (ABNT, 1988) and the associated wind global actions on the model base are used to quantify a structure residual capacity to support accidental horizontal loads.

Since the design and execution processes involve a series of uncertainties, the structure capacity (resistance) may be considered to follow a normal probability distribution, with coefficient of variation equal to 0.20 and the computed structure residual capacity level as the characteristic value correspondent to a 0.95 not to be exceeded probability.

To obtain the characteristic value, the wind direction is assumed horizontal and, for each five floor module, its total action is modeled by a horizontal drag force applied on half of its height. These forces are calculated following the *NBR-6123* (ABNT, 1988) and are dependent on:

- drag coefficient;
- effective frontal area;
- basic wind velocity;
- topographic factor;
- statistic factor;
- soil roughness, building dimensions and height above ground.

Because these factors are different all over the country, the state capital cities are divided into four groups according to the wind basic velocity, V, and calculate the horizontal global wind forces for each structure model inside each group, Tab 2.

Group 1 - V= 30m/s: Aracaju, Belém, Fortaleza, Natal, Recife, Salvador, João Pessoa, Teresina, São Luis, Maceió, Rio Branco, Porto Velho, Macapá and Palmas;

Group 2- V=35m/s: Rio de Janeiro, Boa Vista, Manaus, Cuiabá, Goiânia, Brasília, Belo Horizonte and Vitória;

Group 3-V=40m/s: São Paulo;

Group 4-V=45m/s: Campo Grande, Curitiba, Porto Alegre and Florianópolis.

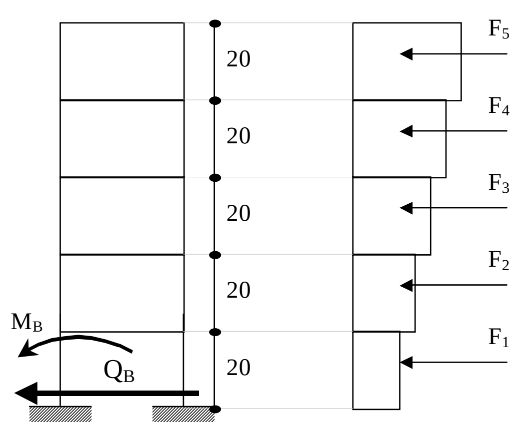

Figure 2. Illustration of wind load application.

Table 2: Brazilian regional global wind forces.

Global forces	Group 1 (kN)	Group 2 (kN)	Group 3 (kN)	Group 4 (kN)
F_1	109	149	194	246
F_2	190	258	337	427
F_3	237	323	421	533
F_4	270	368	481	609
F_5	296	403	526	666

From the previously evaluated forces, F_i, the total shear forces and overturning moments at the base, shown in Tab. 3 are obtained. These quantities are the characteristic values used to define the mean of the structure capacity distribution to accidental horizontal actions. For each city group there are two capacity distributions, one associated to shear and the other related to the overturning moment.

Table 3: Characteristic values of total shear and overturning moment at the base.

Height	Group 1		Group 2		Group 3		Group 4	
	Q_B (kN)	M_B (kNm)	Q_B (kN)	M_B (kNm)	Q_B (kN)	M_B (kNm)	Q_B (kN)	M_B (kNm)
20 m	109	1092	149	1486	194	1941	246	2456
40 m	299	6784	407	9233	531	12060	673	15263
60 m	536	18635	729	25364	953	33128	1206	41928
80 m	806	37568	1098	51135	1434	66788	1814	84529
100 m	1102	64215	1501	87403	1960	114160	24811	144483

Seismic Demand Distribution

The term seismic demand is used to indicate the maximum total structure responses (total shear and overturning moment) under seismic excitation. The seismic ground acceleration is taken as a zero mean second order weakly stationary random process represented by its ground acceleration power spectral density function, $PSD_{AgAg}(\omega)$. In sequence, this function, is propagated through the structure with the relief of the transfer function, $H_{RR}(\omega)$, to obtain the power spectral density function of the structure response, $PSD_{RR}(\omega)$, Eq. (1), Clough and Penzien (1975).

$$PSD_{RR}(\omega) = \left|H_{RR}(\omega)\right|^2 \cdot PSD_{A_g A_g}(\omega) \tag{1}$$

The consideration of the power spectral density function of the structure response allows one to evaluate the time dependent probability distribution function of the demand, which gives the probability distribution of a structure response peak level not to be exceeded along the seismic action duration. This probability distribution is evaluated by considering the first passage problem Eq. (2), according to Vanmarcke (1975); this is a conditional probability given a seismic ground acceleration, a_{max}, since the power spectral density is a function of this parameter.

$$L_T(t^*) = Z \cdot \exp\left(-\alpha \cdot t^*\right) \tag{2.a}$$

$$\alpha = \left(\frac{\dfrac{1}{\pi} \cdot \sqrt{\dfrac{\lambda_2}{\lambda_0}} \cdot \exp\left(\dfrac{-b^2}{2 \cdot \lambda_0}\right) \cdot \left(1 - \exp\left(-\sqrt{\dfrac{\pi}{2}} \cdot \dfrac{b}{\lambda_0} \cdot \left(\sqrt{1 - \dfrac{\lambda_1^2}{\lambda_2 \cdot \lambda_0}}\right)^{1.2}\right)\right)}{\left(1 - e^{\frac{-b^2}{2 \cdot \lambda_0}}\right)} \right) \tag{2.b}$$

$$\lambda_i = \int_w \omega^i \cdot PSD_{RR}(\omega) d\omega \tag{2.c}$$

in which: $L_T(t^*)$ probability of a given demand level not to be exceeded during the interval (0, t*);
- Z probability of starting below the given demand level;
- b a given demand level;
- α decay rate;
- t* stationary ground motion duration;
- λ_i i-th order spectral moment.

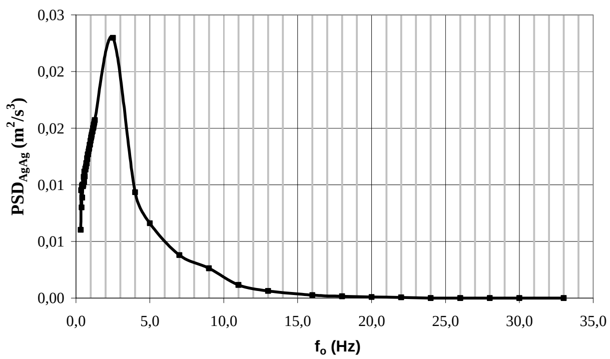

Figure 3. Ground acceleration power spectral density, normalized to a_{max} = 0.1g.

Throughout this work, a $PSD_{AgAg}(\omega)$ under uniform probabilistic association to a general design response spectrum, as given by Almeida (2002), is used. One version of this $PSD_{AgAg}(\omega)$ normalized with respect to a_{max}= 0.1g is shown in Fig. 3. Figures 4 and 5 show the demand conditional distribution for total shear and overturning moment, respectively, in the case of a_{max}= 0.1g. Similar curves for other a_{max} values exhibit the same shapes but they are displaced towards the right along the abscissa, proportionally to increasing total power.

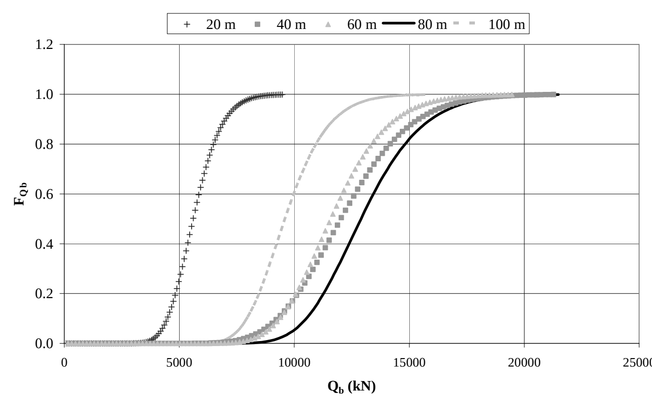

Figure 4. Conditional demand distribution in terms of total shear force at the base, Q_b.

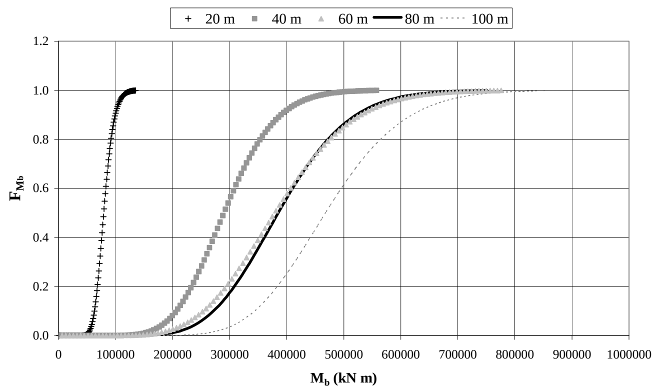

Figure 5. Conditional demand distribution in terms of overturning moment at the base, M_b.

The demand distribution curve differences are a consequence of the earthquake power transfer to the structure response showing that, for a fixed demand level, its not to be exceeded probability, in general, decreases with the structure height.

Now, if the overturning moment power spectral density functions are observed, Fig. 6; it is verified that the concentration of the response power, in general, decreases with decreasing structure height. This justifies the curve relative positions along the horizontal axis in Fig. 5. It is important to note also that the total power is associated with the transfer function amplitude as well as with its distribution in the frequency range, where the earthquake concentrates its power.

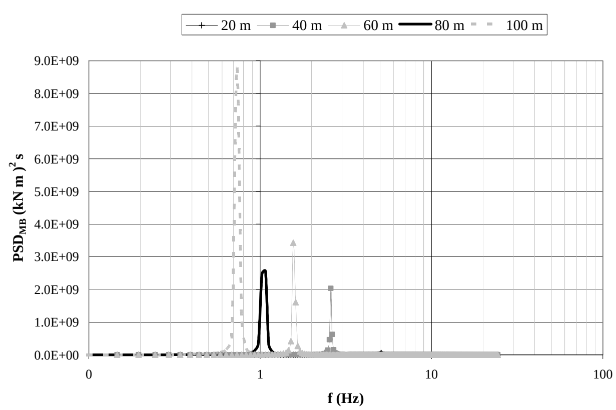

Figure 6. Power spectral density function for overturning moment at the structure base a_{max} = 0.1g.

Hazard Curves

The seismic maximum ground motion acceleration presents uncertainties that are considered by the seismic hazard curves. These curves relate the maximum ground acceleration levels and their not to be exceeded annual probability, $P_h(a_{max})$. An evaluation of seismic hazard is performed by Almeida (2002) for the Brazilian capital cities using the method based on USNRC (1997) which takes under consideration uncertainties in the dimension, location, and frequency of occurrence of earthquakes; in a systematic way, a_{max} is let to vary from 0 to 0.5g. Figure 7 shows the Brazilian seismotectonic provinces and the position of the Brazilian state capital cities, taken from Almeida (2002); also, the seismic hazard extreme curves for these cities are reproduced in Fig. 8.

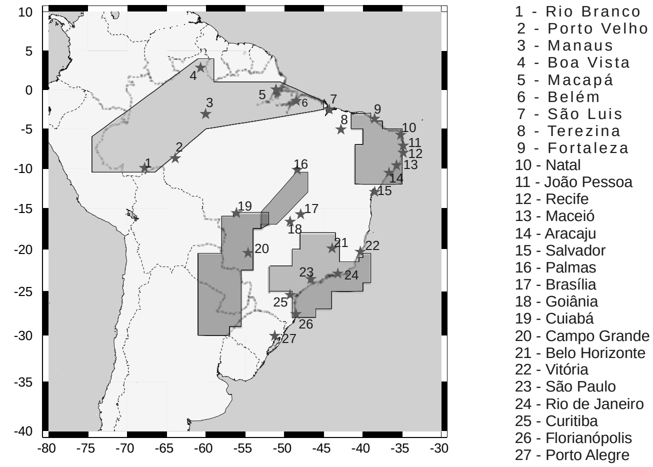

Figure 7. Brazilian seismotectonic provinces and Brazilian state capital cities (Almeida, 2002).

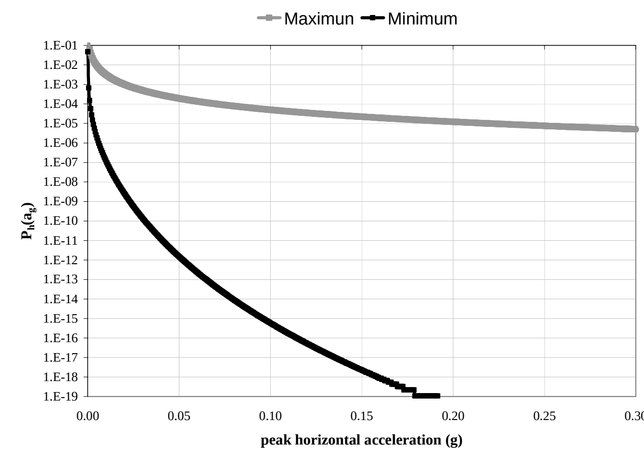

Figure 8. Seismic Hazard extreme curves for Brazilian state capital cities (Almeida, 2002).

Failure Probability

Demand and capacity are random variables dependent and independent of ground motion, respectively. When the seismic demand conditioned to a_{max} exceeds the strength capacity, failure occurs, and the associated conditional probability is $\boldsymbol{P_{f/amax}}$. If demand and capacity are assumed as independent variables, the failure conditional probability is evaluated by convolution of capacity and demand probability density functions, Eq. (3).

$$P_{f\,/\,a\,max} = \int_{-\infty}^{\infty}\int_{c}^{\infty}\left(f_C(c)\cdot f_D(d)\right)dd\,dc$$
$$P_{f\,/\,a\,max} = \int_{-\infty}^{\infty}\left(1-F_D(c)\right)\cdot f_C(c)\,dc \qquad (3)$$

where: $f_C(\ )$ capacity probability density function;
$f_D(\ )$ demand probability density function;
$F_D(\ )$ demand accumulative function;

Finally, the convolution between fragility and hazard curves is established, according to Eq. (4), in order to determine the annual failure probability of the structure models:

$$P_{f\ anual} = -\int_0^{\infty} \left(\left(\frac{dP_h(a_{max})}{da_{max}} \right) \cdot P_{f\ /\ a\ max} \right) da_{max} \qquad (4)$$

where: $P_{f\ anual}$ annual failure probability;

$P_h(a_{max})$ seismic hazard probability;

$P_{f\ /\ a_{max}}$ failure conditional probability, given, a_{max}.

Results

The described methodology is used to evaluate the annual failure probability of a series of structure models when subjected to ground motions compatible to the described seismicity and with the model strength taken as the recommended wind maximum shear and overturning moment at the base. These probabilities, for each state capital, are summarized in Tab. 4 according to the following arrangement:

- four state capital city groups having the characteristic wind velocity as the distinguishing parameter;
- inside the groups, the cities are organized in descending predominant failure probability order.

Table 4 allows to point out the following observations:

- up to 40m height models, the failure probability due to the overturning moment is higher than the one due to base shear; for higher building models base shear dominates instead;
- inside the groups, the failure probability values vary significantly;
- in Tab. 4, the model failure probability values vary from 33×10^{-3} down to 3×10^{-10}, from a 20m high building in Aracaju to a 100m high model in Porto Alegre.

The first point can be understood if it is realized that the wind pressure on buildings increases with height above ground while the seismic rigid forces remain constant; then, relatively, the capacity grows more rapidly than the demand and the overturning moment capacity also grows faster than the base shear capacity.

The next point is explained by the different positioning of the cities inside and outside a seismologic province as well as the relative situation inside the province.

The last observation is the main source of light to clarify the initial questioning in this work. For this purpose, it is necessary to find which would be the allowable maximum failure probability to be considered and this subject may open a long discussion. Instead, it is preferred, in the absence of Brazilian recommendations, to refer to DOE (1994) which prescribes, for conventional structures subjected to seismic loads, a 10^{-3} probability level limit to the annual failure probability. Following this orientation, each city group is subdivided into three regions: the first, Zone I with annual failure probabilities equal or greater than 10^{-2}; an intermediate region, Zone II with probabilities between 10^{-2} and 10^{-3} and Zone III with probabilities lesser than 10^{-3}.

Strictly speaking, Zone I indicates the necessity to have an assisted structure seismic design; Zone II does recommend a design procedure oriented by a couple of specified detailing recommendations and Zone III requires no seismic design concern.

However, a more prudent attitude would be just to keep the discussion inside the limits required to answer the initial question to say: Yes, a seismic resistant design is to be required in many situations.

Table 4: Annual failure probabilities x 10^3.

State Capital City	Structure model height (m)									
	20 m		40 m		60 m		80 m		100 m	
	P_Q	P_M	P_Q	P_M	P_Q	P_M	P_Q	P_M	P_Q	P_M
GROUP 1										
Aracaju	21	**33**	15	17	6.3	5.9	4	2	1.5	1.1
Maceió	20	31	15	16	6.1	5.6	3.8	1.9	1.45	1.1
Natal	14.4	21.4	10.8	11.6	4.8	4.46	3.11	1.60	1.21	0.92
Palmas	15.0	31.0	8.74	10.3	1.98	1.87	0.963	0.35	0.219	0.15
Macapá	9.3	14	6.7	7.3	2.8	2.6	1.7	0.88	0.66	0.5
Belém	9.4	15	6.8	7.4	2.8	2.6	1.8	0.89	0.66	0.5
Recife	8.9	15	6.1	6.8	2.1	2.0	1.2	0.56	0.4	0.29
Rio Branco	8.5	13	6.2	6.8	2.6	2.4	1.7	0.84	0.63	0.47
João Pessoa	8.4	14	5.7	6.3	1.9	1.8	1.1	0.51	0.36	0.26
Porto Velho	6.9	10	5.2	5.6	2.2	2.1	1.5	0.74	0.56	0.42
Fortaleza	7.02	12.7	4.62	5.23	1.50	1.40	0.86	0.385	0.271	0.196
Salvador	1.8	3.3	1.1	1.3	0.33	0.31	0.18	0.068	0.043	0.029
Teresina	0.64	1.3	0.4	0.46	0.1	0.096	0.051	0.017	0.01	0.0064
São Luis	0.078	0.17	0.044	0.053	0.0078	0.0074	0.0029	0.00063	0.00025	0.00013
GROUP 2										
Rio de Janeiro	17	29	12.0	13	4.2	3.9	2.4	1.1	0.73	0.52
Belo Horizonte	17	28	11.0	13.0	4	3.7	2.3	1.0	0.71	0.51
Cuiabá	7.97	12.2	5.86	6.37	2.51	2.34	1.61	0.82	0.61	0.46
Manaus	6.5	11	4.6	5.1	1.8	1.7	1.1	0.54	0.39	0.29
Boa Vista	5.7	9	4.1	4.5	1.7	1.5	1.0	0.51	0.38	0.28
Goiânia	1.38	3.19	0.78	0.943	0.167	0.157	0.0768	0.0252	0.0145	0.00928
Vitória	0.9	2.0	0.5	0.6	0.1	0.1	0.05	0.02	0.01	0.007
Brasilia	6.9	1.6	0.39	0.47	0.076	0.072	0.033	0.0094	0.0048	0.0028
GROUP 3										
São Paulo	11.5	19.7	7.73	8.65	2.57	2.39	1.44	0.60	0.40	0.28
GROUP 4										
Florianó-polis	7.21	12.3	4.87	5.45	1.59	1.48	0.882	0.362	0.241	0.168
Campo Grande	6.42	10.5	4.50	5.0	1.69	1.58	1.03	0.49	0.35	0.26
Curitiba	1.17	2.69	0.66	0.8	0.15	0.138	0.07	0.0248	0.0152	0.0102
Porto Alegre	0.001	0.004	0.0006	0.0008	0.00004	0.00004	0.00001	0.000002	0.0000005	**0.0000003**

Zone I   Zone II   Zone III

P_Q, P_M – annual failure probability of the structure models associated with total shear and with overturning moment at the base.

Conclusion

The Brazilian wind force provisions for conventional buildings insure a structure strength to horizontal actions compatible to a 10^{-2} annual seismic failure probability.

To reach a desirable 10^{-3} level some code provisions must be improved and introduced.

Finally, to insure an annual seismic failure probability level below 10^{-3}, it is required to have a seismic resistant guaranteed design.

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