Macau Regulamento de Segurança e Acções em Estruturas de Edifícios e Pontes

Seismic Action Revision (Consultation Document)

Revision of Chapter IV - Seismic Action - of
Regulamento de Segurança e Acções em Estruturas de Edifícios e Pontes
(RSAEEP), Macau 2008

[Version - 15th of April, 2008]

Chapter IV
Seismic Action

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Chapter IV
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Article 21°

(General considerations)

1. **Introduction**

Macau region consists in one seismic area, qualified as exposed to moderate intensity earthquakes.

The seismic intensity grade and the corresponding expected peak ground acceleration values, considered in present seismic code provisions, are derived from the basic seismic acceleration values and associated risks included in the "China Seismic Ground Motion Parameter Zonation Map" for Macau place, according to the GB 18306 - 2001.

In full compliance with the relevant articles of the Macau RSAEEP – "Regulamento de Segurança e Acções em Estruturas de Edifícios e Pontes", of 1996, these seismic code specifications gives guidance to the characterization and quantification of the relevant design seismic action value and to the definition of the adequate methods to analyze their effects on civil engineering structures.

For the quantification of the Macau seismic actions and the definition of the corresponding seismic response spectra, including local testing and classification of the site ground conditions, a very valuable cooperation of the Guangdong Seismic Institute was provided, through their staff expertise in seismic field research.

2. **Scope**

The code specifications of this Chapter IV of RSAEEP apply to the construction design of buildings or other structures located in the region of Macau, in terms of adequate earthquake-resistant capacity.

> Note: However, some of the present code prescriptions are applicable only to building structures (understood as the ones mainly used by human people) and henceforth called as buildings or building structures along the corresponding code text, as it happens in all the Article 24º content.

The fundamental aim of this earthquake-resistant design is to ensure that, in the event of earthquakes:

- human lives are protected,
- economic damage is limited and,
- important facilities for civil protection remain operational.

The random nature of the seismic events and the limited resources to evaluate their effects are such as to make the fulfilled of those objectives only partially reached and only estimated in probabilistic terms.

Special structures such as offshore structures, dams or other power facilities are beyond of these code provisions.

3. Units and definitions

For the purposes of these seismic code provisions, the SI units system is adopted and the following definitions shall be applied:

- *Building* – construction facility basically to be used directly by people;
- *Return period* or *average recurrence interval of an event* – time period (in years) equal to the inverse of its annual probability of exceedance (expressed in decimal format), or equal to its probability of exceedance (in %) during the life-time (or reference period) of a structure. For example: an event with a return period of 475 years, will occur in average once in every 475 years, and will have an annual probability to be exceeded of 0.002, or 10% of probability of exceedance during an intended life-time of a structure of 50 years;
- *China Seismic Ground Zonation Maps 1 and 2* – official document where it is assigned, also for the Macau region, the seismic input for the following seismic hazard: peak ground acceleration (*PGA*) value and the *spectrum characteristic period* (T_g), for a *ground site-class* type II, with a 10% probability of exceedance in 50 years (equivalent to a return period of 475 years);
- *Reference* or *characteristic peak ground acceleration (PGA)* – defines the seismic hazard on type II *site-class*, and it is derived, for Macau region, from the "China Zonation Map" but adjusted for local geological conditions. It corresponds to the characteristic return period of 475 years, and it is the basis for the determination of the design seismic actions, to be considered in the analysis of structures qualified in ordinary importance category;
- *Spectrum characteristic period,* T_g – spectral period parameter of the response spectrum curve, depending of the *ground site-class* type, where the spectrum curve starts its decay section;
- *Importance category* of a building or structure – qualification of structures according to the social-economic importance of their using functions. Higher importance levels should correspond to lower severity grades of after-shake damaging effects. Reflected through the corresponding *importance factor*;
- *Acceleration response spectrum* – is a plot of the maxima of the acceleration response for a single degree of freedom (SDOF) system with various fundamental vibration periods, when subjected to an earthquake ground motion;
- *Reduced-elastic or design response spectrum for non-linear-elastic response* – is an elastic spectrum corresponding to a target of inelasticity and is estimated by dividing the ordinates of the elastic spectra by the *behavior factor (q)*. Is to be used in the assessment of the seismic effects in the ultimate (or no-local-collapse) limit state, and so called as *design spectrum*;
- *Behavior factor (q)* – is the quantification of the inelastic behavior of structures, closely related to their ductility performance. The value q is the ratio of the seismic forces that a structure would experience if its response was full elastic with 5% damping, to the (smaller) seismic yield forces that could occur in the system, and so enabling some grade of inelastic deformation occurrence. This is equivalent to reduce the seismic elastic values induced in structures (or the elastic response spectrum ordinates),

dividing them by a *behavior factor (q)* and then performing an explicit elastic analysis. It should be pointed out that high values of the *behavior factor (q)* correspond to large inelastic deformations, normally equal to the ones derived from the linear analysis, multiplied by the above defined *behavior factor (q)* Otherwise, if non-linear analysis methods were used, then both the effective internal forces and displacements are taken to be equal to those directly derived from the non-linear analysis, therefore without the intervention of the *behavior factor(q)*;

- *Reduced-elastic response spectrum for serviceable verification* – is the *design response spectrum* multiplied by the *reduction factor* (ν) to be used in the assessment of the seismic effects in the serviceable limit state (or damage limitation).
- *Reduction factor* (ν) – factor to be applied to the characteristic seismic action in order to obtain the *serviceability seismic action* (for damage limitation).

4. **<u>Reference codes</u>**

According to the general principles of structural safety, where the present RSAEEP of Macau is based, also these seismic code provisions are following closely some concepts of the EN (European codes), in this case the "Eurocode 8: Design of structures for earthquakes resistance", version EN 1998-1:2004.

However, the source for the definition of the seismicity and peak ground acceleration, *PGA*, assigned to the Macau place, was the "China Seismic Ground Motion Zonation Map", published with the GB 18306-2001.

Several concepts of the GB 50011-2001 – "Code for Seismic Design of Buildings" were followed in the definition of the Macau seismic response spectrum curves, namely on their site geologic adjustment of some key parameters.

Article 22°

(Performance requirements and conformity criteria)

1. **<u>Fundamental requirements</u>**

Only one seismic area is considered in Macau region due to the small size of the territory and to the uniformity of the local seismic data records. This is consistent with the "China Seismic Ground Motion Zonation Map" where, for Macau place, only one seismic intensity and one peak ground acceleration value *(PGA)* are assigned.

Following the updated Eurocode 8 concepts, the present RSAEEP seismic code provisions provide for a two-level seismic design of buildings or other civil engineering structures, with the following two explicit performance levels for the requirements to be reached, each one with an adequate degree of reliability:

- **No-collapse requirement**: protection of life under a rare or no-frequent seismic action, through prevention of partial or global collapse of a structure, or its parts, and retention of structural integrity and residual load capacity after the event. This implies that the structure is significantly damaged, presents some moderate permanent drifts, but retains its full vertical load-bearing capacity and enough residual lateral strength and stiffness to protect life, even with strong aftershocks. However, its repair may be costly onerous or even uneconomic.

- **Damage limitation requirement**: mitigation of property losses, and the maintenance of their specified service demands, under frequent seismic actions, through reduction of structural and non-structural damage. This implies that the structure itself does not present permanent drifts, its elements have no permanent deformations, retain their strength and stiffness, and do not need repair. Non-structural elements may be slightly damaged, but easy and economic repair could be done afterwards.

The no-collapse performance level corresponds to moderate but no-frequent earthquakes where the drift of the structure is close to, or has already reached, the elasto-plastic limit in some few vertical elements, but the whole structure do not collapse. The damage limitation performance level corresponds to weak/moderate and more frequent earthquakes and the ductile structure is full in elastic state.

Under the RSAEEP safety verification concepts of structures, the two explicit performance levels – collapse prevention and damage limitation – are pursued under two different seismic actions. The seismic action under which collapse should be prevented corresponds to the *design seismic action*, and the one under which damage limitation is pursued corresponds to a *serviceability seismic action*.

The buildings or structures are classified in four categories, A to D, according to the social-economic importance of their using functions (such as high-occupancy buildings, schools, public assembly halls, museums, etc.) and the importance of their use as essential for civil protection in post-earthquake period. The Table IV.1 describes each importance category and defines the corresponding *importance factors* (γ_I).

For buildings of ordinary importance (category C with γ_I =1.0) the following two seismic actions are defined, in terms of design objective, risk of exceedance probability and return period:

- a *design seismic action* (for collapse prevention) with 10% exceedance probability in 50 years (characteristic return period: 475 years);
- a *serviceability seismic action* (for damage limitation) with 10% exceedance probability in 10 years (return period: 95 years).

For these ordinary importance buildings (γ_I =1.0) the above defined *design seismic action* is equal to the reference or characteristic seismic action, corresponding to the reference peak ground acceleration value (*PGA*) assigned to Macau seismic zone (see Article 23º nº.2). For the same ordinary importance buildings the *serviceability seismic action* could be obtained applying the *reduction factor* ν=0.4 (shown in Table IV.1) to the *design seismic action* used for the collapse prevention, which is equivalent to decrease the return period from 475 to 95 years.

Table IV.1 – Importance categories for buildings and structures

<table>
<tr><th>Importance Category</th><th>Building and Structural Cases</th><th>Importance Factor (γ_I)</th><th>Reduction Factor (v)</th></tr>
<tr><td>D</td><td>Buildings of minor importance for public safety, e.g. agriculture and storage facilities, etc.</td><td>0.4 - 0.8</td><td rowspan="2">0.4</td></tr>
<tr><td>C</td><td>Ordinary buildings, not belonging to other categories.</td><td>1.0</td></tr>
<tr><td>B</td><td>Buildings whose seismic resistance is of importance in view of the consequences associated with collapse, e.g. schools, assembly halls, stadium, cultural institutions, etc.</td><td>1.2</td><td rowspan="2">0.5</td></tr>
<tr><td>A</td><td>Buildings whose integrity against collapse or damage limitation have vital importance for civil protection during and post-earthquakes, e.g. hospitals, fire stations, power plants, etc.</td><td>1.4</td></tr>
</table>

For buildings or structures of importance categories other than category *C* the corresponding *importance factor* (γ_I) or the *reduction factor* (v) shall be applied to the characteristic seismic action values, respectively, to verify the ultimate limit state or the serviceable limit state, as described in the next n^o. 2.

2. Conformity criteria

a. Verification for no-collapse (ultimate limit state)

Ultimate limit states compliance are associated to prevent collapse or other forms of structural failure which might endanger the safety of people. It shall be verified if the structure has the required resistance and energy-dissipation capacity.

The resistance and energy-dissipation capacity to be assigned to the structure are related to the extent to which its non-linear response is to be exploited. In operational terms such balance between resistance and energy-dissipation capacity is characterized by the values of the *behavior factor (q)* and the associated ductility classification, which are usually given in the relevant structural materials codes. In general, however, the ductility classification shown in Table IV.2 and the corresponding *behavior factor (q)* are suggested to be used in the structural design.

Table IV.2 – Recommended values for the *behavior factor (q)*

Structural behavior model	Structural ductility class	*Behavior factor (q)*
Low dissipative structural behavior	Low	≤ 1.5
Dissipative structural behavior	Medium	< 3
High dissipative structural behavior	High	See Structural Steel and Reinforced Concrete Codes, or other bibliography

Note: The values the *behavior factor* (q) usually also account for the influence of the viscous damping being different from 5%.

The relevant *importance factors (γ_I)*, shown in Table IV.1, shall be applied to the characteristic seismic action, and this should be done, as described later in Article 23º, nº.2, c, acting only in the relevant RSAEEP accidental combination.

In the analysis it shall be taken into account the possible influence of second order (P-Δ) effects on the values of the action effects.

It must be call to attention that the foundation system deformability should be considered in the assessment of the effects of soil-foundation-structure interaction.

b. Verification for damage limitation (serviceable limit state)

The damage limitation requirement for buildings is simply an upper limit verification on the interstorey drift ratio demand under the more frequent (or lower return period) *serviceability seismic action* as defined in nº.1 of this article.

The *serviceability seismic action* is defined by multiplying the entire *reduced-elastic or design spectrum for non-linear-elastic response* (see Article 23º nº.2.b.(4)) by the relevant *reduction factor* (see Table IV.1) obtaining the *reduced-elastic response spectrum for serviceable verification*. And then the values of the displacements to be used to calculate the drift ratios are those directly obtained from the analysis with the *reduced-elastic response spectrum for serviceable verification* multiplied by the *behavior factor, q*.

The highest interstorey drift ratio determined shall not exceed the following upper limits:

- 0.5% - for buildings having brittle non-structural elements (e.g. masonry partitions) attached to the structure**,**
- 0.75% - for buildings having ductile non-structural elements (e.g. composite partitions) attached to the structure,
- 1% - for buildings having non-structural elements fixed in a way so as not to interfere with structural deformations, or in absence of non-structural elements.

As an alternative to the previous direct assessment of the displacements for the verification of the damage limitation the following method could be used:

$$\text{Interstorey drift ratio} = d_r\, v\, /\, h$$

where:
- d_r is the design interstorey drift from the analysis with the *design seismic action*, *d*, multiplied by the *behavior factor* ($d_r = d\, q$);
- v is the *reduction factor* which takes into account the lower return period of the seismic action associated with the damage limitation requirement;
- h is the storey height.

Article 23°

(Ground conditions and seismic action)

1. **<u>Soil ground conditions in construction sites</u>**

   a. **General**

   The site soils qualification, under the point of view of the increasing severity of their seismic effects on buildings and structures, varies from favorable to unfavorable and hazardous. This increasing severity condition corresponds to:

   - the more “favorable site soils” correspond to: sound rock or dense and homogeneous medium-stiff soils in wide-open areas,
   - the “unfavorable site soils” correspond to: soft-clay soils, liquefied soils, silt soils, recently back-filled soils, stratified and heterogeneous rock formations, boundary of slopes, river beds, fractured zones of faults, etc;
   - the “hazardous site soils” correspond to: locations where landslide, subsidence or fracture are possible to occur during earthquakes or close to active faults.

   The location of the construction sites and the nature of the supporting ground should privilege the favorable ground conditions, always minimizing the risks of ground rupture, slope instability and permanent settlements caused by liquefaction or densification due to earthquakes. Any possibility of occurrence of these unfavorable ground conditions shall be always carefully investigated under experimented and specialized geologic studies. The construction on hazardous ground condition shall not be allowed.

   The above general principles and descriptions require a *site-class* code qualification of the geotechnical site conditions where the buildings and structures are to be founded. Therefore, the resultant quantification of the site seismic actions (represented by response spectra) is closely depending of that site soil profile classification, which is based on a prudent site evaluation and interpretation of the relevant soil parameters, such as shear-wave velocities, standard penetration testing results (SPT) and thickness of overlaying layers, in each site soil profile.

   b. **Qualification of construction site ground conditions**

   (1) Objective: Four types of *ground site-class* (I, II, III and IV) have been selected to qualify the site soil profiles, as described in the Table IV.3 and Table IV.4, where the corresponding site soil profiles and geologic parameters are described.

   This soil classification could be used to consider the site ground conditions influence on the quantification of the seismic action. As described later in next specification n^{o}.2, from the *site-class* classification is possible to obtain the important parameter *spectrum characteristic period*, T_g, through the Table IV.5, and so to define the elastic response spectrum to be used for each site ground condition.

   (2) Geological prospection: As a general design guideline, to define the adequate *ground site-class* for a seismic resistant structure, it is prudent to perform a local soil investigation based in borehole sampling that should include standard penetration testing (N_{SPT}) and shear-wave velocity (v_s) testing for each site soil layer, to allow the definition of the corresponding soil profiles, according to the criteria shown in the following Tables IV.3 and IV.4.

Note: An alternative option, not requiring shear-wave tests, could be followed based on a experienced interpretation of the correlation between the shear-wave velocity, v_s , and the more commonly available N_{SPT} value, for each type of soil layer.

Only for building structures assigned to lower importance categories, the categories C and D (see previous Table IV.1), with less than 10 storeys and no more than 30m height, if shear-wave velocity test results and N_{SPT} values are not available for each type of soil layer, then the following Table IV.3 could be used to estimate the shear-wave velocity of each soil layer, and proceed the selection of the corresponding *ground site-class*, presented in Table IV.4, based only in an experienced interpretation of the available geotechnical soil description.

Table IV.3 – Classification and description of soil range related to their shear-wave velocity , v_s , and N_{SPT} values

Type of soil layer	**Geotechnical description**	**Shear-wave velocity, v_s, of a soil layer (m/s)**	**N_{SPT}**
Rock or stiff soil	Sound rock, rock-like formations, or deposits of very dense sand, gravel, very stiff clay (about more than 20 meters of thickness), with a gradual increase of mechanical properties with depth	$v_s > 500$	-- > 70
medium-stiff soil	Medium dense to slightly dense deep deposits, dense to medium-dense gravel, coarse to medium sand or stiff clay and cohesive soil	$500 \geq v_s > 250$	70-25
medium-soft soil	Slight dense gravel, coarse to medium sand, fine to mealy sand other than that which is loose, soft-to-firm cohesive soil, silt, sand landfill	$250 \geq v_s > 140$	25-10
soft soil	Mud soil, loose sand, new alluvial sediment of cohesive soil or soft clays/silts, soft landfill	$v_s \leq 140$	< 10

Note 1: N_{SPT} evaluated by standard penetration testing and v_s , the shear-wave velocity in m/s, evaluated by direct site tests or estimated by correlation to the available N_{SPT} values.

Note 2: Deposits including either thick layers of saturated soft clays/silts with high plasticity index, or liquefiable soils, are not consider here as they could lead to severe failures.

(3) Methodologies for the classification of ground sites: The geologic ground conditions on construction sites are represented usually by the corresponding site soil profile, which is classified in one of the four *site-classes* defined in Table IV.4.

This classification depends on the *thickness of overlaying layer*, d_e**,** and the site *equivalent shear-wave velocity* value, v_{se}, corresponding to the above mentioned soil site profile. These two values are defined as follows:

(a) The *thickness of site overlaying layer*, d_e in m, shall be determined according to the following rules:

- In general is the distance from the ground surface to a soil layer level, under which the shear-wave velocity is v_s >500m/s (such as a rock or a stiff-soil layer).
- If a soil layer, with a depth, d_1, below the surface level less than 5m, and its shear-wave velocity, not less than 400m/s, is more than 2.5 times the shear-wave velocity of the layer above, then the *thickness of site overlaying layer*, d_e, could be assumed as the distance from the ground surface to this layer ($d_e = d_1$).

(b) The *equivalent shear-wave velocity*, v_{se} in m/s, of the soil profile shall be calculated according to:

$$v_{se} = d_0 / t$$

where: d_0 *calculated depth* of the overlaying layer:
d_0 = 20m (if d_e> 20m) or $d_0 = d_e$ (if $d_e \leq$ 20m)

t transmission time of the shear-wave from the ground surface to the calculated depth (d_0), calculated as:

$$t = \sum_{i=1}^{n} (d_i / v_{si})$$

d_i thickness of i-th soil layer within the range of calculated depth (d_0), in m;

v_{si} shear-wave velocity of the i-th soil layer within the calculated depth (d_0), in m;

n number of soil layers within the range of calculated depth d_0.

Table IV.4 – *Ground site-class* classification of the site soil profiles

***Equivalent shear-wave velocity - v_{se}* (m/s)**	***Ground Site-class* of the site soil profile**			
	I	**II**	**III**	**IV**
v_{se} > 500	0			
500 ≥ v_{se} > 250	<5	≥5		
250 ≥ v_{se} >140	<3	3 - 50	>50	
v_{se} ≤ 140	<3	3 - 15	>15 -80	>80

Note: The values of shear-wave velocities for each site i-th layer (v_{si}) could be obtained by direct site testing or, if these values are not available, the correlated N_{SPT} values could be used to estimate the shear-wave velocities, and the same v_{se} expression could be used to determine the relevant *equivalent shear-wave velocity* for the site soil profile.

2. <u>Seismic action: quantification and representation</u>

a. Seismic zone and assignment of its peak ground acceleration seismic value

For the Macau seismic area, the seismic hazard is defined through the value of the reference *PGA*, with the parameter a_{gR}, derived from the already mentioned "China National Zonation Map" as *0.10g*, corresponding to a *ground site-class* II and with a probability of exceedance of 10% in a reference period of 50 years. However after local geological analysis performed in the Macau region it was assumed a more accurate reference peak ground acceleration, $\boldsymbol{a_{gR}}$**,** of *0.123g*.

This reference *PGA* value assigned to Macau as *0.123g* corresponds to the *design seismic action* for the no-collapse requirement of an ordinary structure ($\gamma_I = 1.0$) localized in a ground site class II.

For other *importance category* of structures (or other return periods) the *design seismic action* shall be multiplied by the corresponding *importance factor* (see TableIV.1 and the following n°.2-b).

From these general concepts results the definition of the response spectra presented in item b.(2), namely the two following parameters:

- the *coefficient of influence* value for the horizontal seismic action shall be assigned to α_{max}= 0.30 (see the definition in n°.2-b.(2) in this article) corresponding to a reference *PGA* of $\boldsymbol{a_{gR}}$ = *0.123g* m/s^2 for earthquake events with a probability of exceedance of 10% in 50 years or a return period of 475 years, to be used in the definition of the relevant response spectra;
- the *spectrum characteristic period,* T_g, depends on the *ground site-class* type, as shown in Table IV.5.

b. Basic representation of the seismic action

(1) General – On these code specifications the earthquake motion at a given point on the ground surface and induced to the base of a structure located there is represented, in terms of structural dynamics, by an elastic ground acceleration response spectrum. Where in the ordinates of the spectrum are given the maximum elastic acceleration response by, in abscises the fundamental vibration period of the structure. This spectrum will be henceforth designated as elastic response spectrum.

The basic shape of the elastic response spectrum is taken as being the same for the no-collapse requirement (ultimate limit sate – *design seismic action*) and for the damage limitation requirement (serviceable limit state – *serviceability seismic action*).

The horizontal seismic action is described by two orthogonal components assumed as being independent and represented by the same horizontal response spectrum.

The GB 50011-2001 provisions were followed to define the relevant seismic response spectrum curve, in close interpretation of the reference peak ground acceleration value considering the *ground site-class* effective conditions, the fundamental vibration period and the damping of the structure.

(2) Horizontal elastic response spectrum – For the horizontal components of the seismic action, the elastic response spectrum curve $\boldsymbol{S_e(T)}$, normalized by the acceleration of gravity (*g*), is defined by the following expressions (see Figure IV.1):

$$0 \le T \le 0.1(s): \qquad S_e(T) = \alpha_{\max}[0.42 + \frac{T}{0.1}(\eta_2 - 0.42)]$$

$$0.1(s) < T \le T_g: \qquad S_e(T) = \alpha_{\max}\eta_2$$

$$T_g < T \le 5T_g: \qquad S_e(T) = \alpha_{\max}\eta_2\left(\frac{T_g}{T}\right)^{\gamma}$$

$$5T_g < T \le 6(s): \qquad S_e(T) = \alpha_{\max}[\eta_2 0.2^{\gamma} - \eta_1(T - 5T_g)]$$

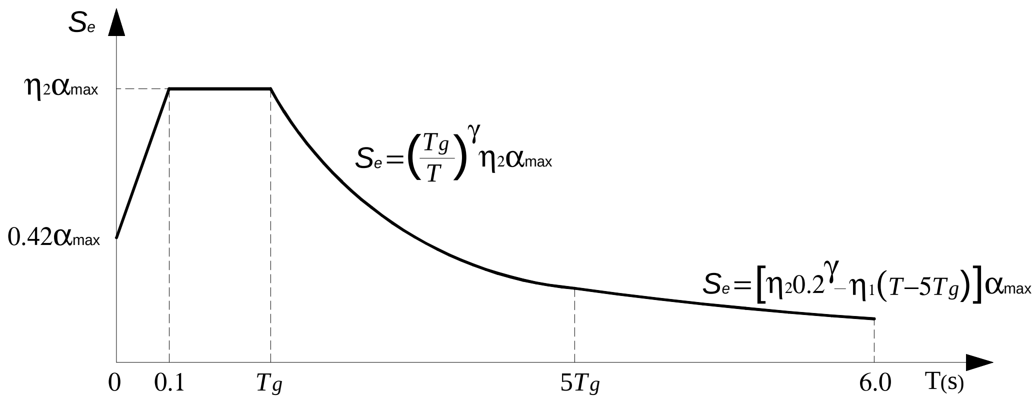


Figure IV.1 – Horizontal Elastic Response Spectrum Curve - $S_e(T)$, normalized by the acceleration of gravity

where: α_{max} *coefficient of influence* value for the horizontal seismic action, determined for Macau as follows: $\alpha_{max} = \frac{a_g}{g}\beta_{\max}$,

where: a_g *PGA* parameter ($a_g = a_{gR}\,\gamma_I$);

g acceleration of gravity (g = 9.8m/s^2);

$\beta_{\max}$ magnification factor taken as $\beta_{\max}$=2.4 according to the geological site conditions of Macau;

So, if the *importance factor* (γ_I) is equal to 1.0 then $a_g = a_{gr}$ = 0.123g and the α_{max} value is equal to 0.30 .

For the verification on damage limitation (or serviceability limit state) this value α_{max} shall be multiplied by the relevant *reduction factor* (v), (see Table IV.1).

In cases where, by exception, a building needs to be built in a very unfavorable site soil conditions (such as lonely hills, non-rocky steeps or boundary of slopes), an amplification factor could be applied to this α_{max} value.

T_g *spectrum characteristic period* defined in the following Table IV.5:

Table IV.5 – *Spectrum characteristic period*, T_g, for the several *ground site-class*

Site-class	**I**	**II**	**III**	**IV**
T_g **(s)**	0.35	0.45 (0.65)	0.65 (0.85)	1.10

Note: The values of T_g in blankets represent the *spectrum characteristic period* in saturated mud area (plasticity index >19).

ζ damping ratio, in decimals;

γ, η_1 and η_2 adjustment factors of the spectrum curve for the different values of the damping ratio, according to the following expressions:

$$\gamma = 0.9 + \frac{0.05-\zeta}{0.5+5\zeta}$$

$$\eta_1 = 0.02 + \frac{0.05-\zeta}{8} \quad \text{(if } \eta_1 < 0\text{, then } \eta_1 = 0.0\text{)}$$

$$\eta_2 = 1 + \frac{0.05-\zeta}{0.06+1.7\zeta} \quad \text{(if } \eta_2 < 0.55\text{, then } \eta_2 = 0.55\text{)}$$

(3) Vertical elastic response spectrum – The vertical component of the seismic action is represented by the vertical elastic response spectrum curve, $\boldsymbol{S_{ve}(T)}$, directly derived from the horizontal elastic response spectrum curve, $\boldsymbol{S_e(T)}$, reducing namely their α_{max} and T_g parameters in 50%.

This reduction in Macau region would result in a design peak ground vertical acceleration (vertical *PGA*) near to 0.06g, much lower than the 0.25g value, beyond which the vertical component of seismic action could be important, and so to justify why, in this region, it is not required to consider this vertical component in the earthquake-resistant design of structures.

Note: In addition it should be noted that the fundamental vibration period of vertical vibration of structures is controlled by the axial stiffness of vertical members and is short, and so spectral amplification of vertical ground motion is small. However this does not occur if long span structural beam, slab or cantilever elements, or beams supporting columns, or horizontal prestressed beams exist.

Therefore, these two conditions shall be complied, together, to conclude that the vertical seismic component is probably required to be taken into account:

- design peak vertical acceleration value higher than 0.25g, and
- horizontal elements with significant span values exist in the structure.

(4) Reduced-elastic or design spectrum for non-linear-elastic response – The capacity of structural systems to resist seismic actions in a non-linear range normally allows their design for resistance to seismic forces smaller than the ones corresponding to a full linear elastic response.

To avoid the use of explicit inelastic structural analysis methods in the design the capacity of structures to dissipate energy (through its own ductile behavior or other devices) could be taken into account if an elastic analysis is performed, based on a reduced-elastic response spectrum. This reduction could in general be obtained by introducing a *behavior factor (q)* with values related to that ductile ability, as suggested in Table IV.2. Similar *behavior factors* (or equivalent) could be found in the relevant structural materials codes, such as the ones governing the design of reinforced concrete or steel structures.

For the horizontal component of the seismic action the reduced-elastic (henceforth called as design spectrum), $\boldsymbol{S_d(T)}$, shall be defined by the following expressions:

$0 \le T \le 0.1(s)$: $\quad S_d(T) = \alpha_{max}[0.28 + \frac{T}{0.1}(1.0/q - 0.28)]$

$0.1(s) < T \le T_g$: $\quad S_d(T) = \alpha_{max}/q$

$T_g < T \le 5T_g$: $\quad S_d(T) = \alpha_{max}/q\left(\frac{T_g}{T}\right)^{0.9}$

$5T_g < T \le 6(s)$: $\quad S_d(T) = \alpha_{max}/q[0.2^{0.9} - 0.02(T - 5T_g)]$

where: q *behavior factor*;

T_g *spectrum characteristic period*, depending of the *ground site-class* type (see Table IV.5).

Note: The chosen values of the behavior factor *(q)* also account for the influence of the damping being different from 5% and then, in these reduced expressions of the reduced-elastic or design spectrum, the values of the parameters η_1, η_2 and γ of the elastic response spectrum are replaced by: $\eta_1 = 0.02$, $\eta_2 = 1.0$ and $\gamma = 0.9$ (see n°.2.b.(2) in this article).

The design spectra given by these equations are not adequate to design structures with base-isolation or other energy dissipation devices. For these cases special studies and methods are required to derive the spectra to be used in structural design.

c. Combination of seismic action with other actions

The design value of the seismic action, S_{Fa}, shall comply with the Articles 5º to 10º of the Macau RSAEEP, where the corresponding combinations are defined. Although, as present along this seismic code, a new *importance factor* (γ_I) as been introduced in the accidental combination for the accident action being an earthquake, as follows:

$$S_d = \sum_{i=1}^{m} S_{Gik} + \gamma_I . S_{Fa} + \sum_{j=1}^{n} \psi_{2j} S_{Qjk}$$

The effects of the seismic action shall be assessed by taking into consideration the presence of the masses associated with all gravity loads corresponding to the permanent loads ($\sum G_k$) and the almost permanent value of the variable loads ($\sum \psi_2 Q_k$).

Article 24°

(Design of building structures - Methods of analysis)

1. <u>Introduction</u>

This article covers the general specifications for the earthquake-resistant design of buildings and shall be used in conjunction with the previous Articles 21º, 22º and 23º.

2. Earthquake-resistant characteristics on building structures

a. Basic principles

Although the seismicity of this region could be assumed as moderate, the principles in which are based the earthquake resistant concepts in building structures are maintained as applicable, thus enabling the achievement of the structural systems under an enlarged scope of induced actions.

The principles to be considered, soon at the conceptual phase, should include the following target concepts: structural simplicity, bi-directional regularity and symmetry, translational and torsional resistance and rigidity, diaphragmatic behavior at the storey level and adequate foundation.

In the following item (b) the structural regularity in plan and elevation, of building structures, are defined and described on their effects in structural modelization and analysis methodologies choices.

b. Criteria for structural regularity

In these specifications, the earthquake-resistant design of non-regular building structures is not forbidden, but only the use of regular conditions are strongly encouraged, due to (1) reasons of simplicity to produce adequate and reliable design studies, (2) reasons of economy, taking advantage of the ductility of structural materials, and (3) to minimize severity and risks of damage occurrences, resulting from the hazardous nature of earthquakes.

It is evident, from the damage observation after earthquakes that regular building structures tend to behave much better than the irregular ones. However a precise definition of this regularity in terns of seismic response of structures stays not yet clear in the more advanced international seismic codes.

Based on these difficulties, these specifications do not establish strict rules to distinguish regular from non-regular building structures and rather it provides some set of characteristics that a structure should possess to be classified as regular.

In these terms, and following some concepts suggested in EN.1998-12004, it is recommended, in Appendix A to Annex 4, a criteria for the classification of regularity in plan and in elevation applicable to building structures.

This classification aims to provide qualitative and preliminary criteria to choose more or less simplified structural models and the more adequate methods of analysis. Effectively this regularity distinction affects several aspects of the seismic design, such as:

- the structural model to be analyzed, which can be either a composition of several planar models, in case of regularity in plan, or a spatial model only, in case of non-regularity in plan;
- the method of analysis, which can be a simplified response spectrum static analysis (lateral force static method), in case of regularity in elevation, or a modal response spectrum dynamic analysis, in case of non-regularity in elevation;

- the value of the *behavior factor (q)*, which shall be decreased for structures non-regular in elevation (values of q in Table IV.2 shall be multiplied by 0.8).

The implication of the structural regularity on above aspects could be summarized in the following Table IV.6.

Table IV.6 – Structural regularity versus seismic analysis criteria

Structural Regularity [a]		Structural Model	Method of linear elastic analysis	Behavior factor *(q)*
Plan	Elevation			
Y	Y	Planar	Static Lateral Force [b]	Reference value
Y	N	Planar	Dynamic Modal	Decreased value
N	Y	Spatial	Static Lateral Force [b]	Reference value
N	N	Spatial	Dynamic Modal	Decreased value

Notes: (a) - according to Appendix A to Annex 4;
(b) - if the conditions $T_1 \leq (2.0s$ and $4T_g)$ are satisfied (Article 24º nº.3.c.(1))

It is important to note that the torsional effects in structures, if they exist due to structural irregularities in plan, are full considered by the use of spatial static or dynamic methods of analysis.

3. Structural analysis

a. General

Within the scope of these specifications the seismic effects on a building structure shall be determined assuming, for analysis purpose, that its behavior is full linear, and so enabling an explicit use of linear methods of analysis.

As a reference criterion, the method for determining the seismic effects in structures shall be the modal response spectrum dynamic analysis, which is applicable to all types of structures, assuming a linear-elastic model of the structure.

However, depending on the structural regularity characteristics of each case, as defined in the nº.2 of this article, a simplified lateral force static method of analysis could be used, if the conditions detailed in Appendix A of Annex 4 are verified.

Note: Both of the mentioned linear methods of analysis (static or dynamic) use the reduced-elastic spectrum $S_d(T)$ defined in Article 23°, nº.2.b.(4), which is essentially the elastic response spectrum divided by the *behavior factor* (*q*), and so assuming some limited non-linear response of ductile structures.

The masses to be considered in the methods purpose for seismic analysis shall be assessed according to what is exposed in Article 23º n.º2.c.

Although these code specifications do not give guidance for the alternative use of non-linear methods, such as: the non-linear static analysis and the non-linear time-history dynamic analysis.

b. Accidental torsional effects

In both regular and irregular structures the center of mass of each floor shall be considered displaced from its nominal calculated location, in each direction, to account for the possible torsional effect from the seismic ground motion and the uncertainties in the location of masses, by an accidental eccentricity given by: $e_{ai} = \pm 0.05 L_i$

where: e_{ai} accidental eccentricity of storey mass i from its nominal location, applied in the same direction at all floors;

L_i the floor-dimension perpendicular to the direction of the seismic action.

c. Simplified lateral force method of static analysis

(1) General

To be used as a simplified method to design building structures, this type of linear static analysis is performed under a set of lateral forces in two independent orthogonal directions (x and y). In case of structural regularity in plan the analysis can be processed by composition of two independent planar analysis, one for each main horizontal direction. In case of no plan regularity the structure shall be analysed under a spatial structural model.

However this static simplification can only be applied if the building structural response is not significantly affected by contributions from modes of vibration higher than the fundamental mode, in each horizontal direction of analysis. This requirement is satisfied if the following two conditions are met:

- the fundamental vibration periods, T_1, in the two directions has to be smaller than 2.0*s* and $4T_g$ (the *spectrum characteristic period*, defined in Article 23º nº.2.b.(2) and Table IV.5);
- the structure has to be regular in elevation.

(2) Base shear force

The expression to determine the base shear force, F_b, for the two horizontal directions in which the structure will be analysed is as follows:

$$F_b = S_d(T_1).G.\lambda$$

where: $S_d(T_1)$ the ordinate of the design response spectrum normalized by the acceleration of gravity (g) at the fundamental vibration period of the structure in the horizontal direction of interest (T_1);

G the total gravity load of the building associated with the permanent loads ($\sum G_k$) and the almost permanent value of the variable loads ($\sum \psi_2 Q_k$) above the foundation or above the top of a rigid basement;

λ a correction factor, the value of which is equal to: λ=0.85 if $T_1 \leq 2T_g$ and the building has more than two storeys, or λ=1.0 otherwise.

For the determination of the building fundamental vibration period, T_1, for the two principal directions, some formulation is presented in Appendix B to Annex 4.

(3) Distribution of the horizontal seismic forces

To assess the effects of the seismic action in the structures the base shear force should be distributed along height of the structure by horizontal forces F_i applied to each storey level. These horizontal forces *Fi* are given by:

$$F_i = F_b . \frac{z_i . G_i}{\sum z_j . G_j}$$

where: F_b the base shear force;

G_i G_j the storey gravity loads associated with the permanent loads ($\sum G_k$) and the almost permanent value of the variable loads ($\sum \psi_2 Q_k$);

z_i z_j the heights of the gravity loads G_i G_j above the level of application of the seismic action (foundation or top of a rigid basement).

It should be noted that this formulation for the distribution of the base shear force over height assumes that the horizontal displacements of the fundamental mode shape increase linearly.

These horizontal forces F_i shall be distributed to the lateral load resisting system assuming the floors are rigid in their plane. Or, the masses and the moments of inertia of each floor may be lumped at the centre of gravity (or mass) and the lateral forces could be applied there, instead.

(4) Accidental torsional effects

As an alternative to the concept of displacing the centre of masses (see n^o.3.b. of this article) in this method the accidental eccentricity can also be considered by multiplying the action effects (the horizontal seismic components - F_i), by a factor δ given by:

$$\delta = 1 + 0.6\, x/L_e.$$

where: x distance of the element under consideration from the center of mass of the building plan, measured perpendicularly to the direction of the seismic action considered;

L_e distance between the two outermost lateral load resisting elements, measured perpendicularly to the direction of the seismic action considered.

If the analysis is performed using two planar models, one for each main horizontal direction, the accidental torsional effects may be determined by doubling the accidental eccentricity - $e_{ai} = \pm\, 0.10 L_i$ - and for this case the factor δ shall be equal to $1 + 1.2\, x/L_e$.

d. Modal response spectrum dynamic analysis

(1) General

This method is a dynamic method of analysis based on a linear-elastic behavior concept of the structures. Mainly it shall be used in cases where the simplified static method is not applicable, such as for highly irregular buildings in elevation (see Table IV.5).

The dynamic method includes the modal superposition of the relevant modal contributions. The response of all modes of vibration, contributing significantly to the global response of the structure, shall be taken into account. And this may be deemed to be satisfied if either one of the following conditions are complied:

- the sum of the effective modal masses, for the modes to be taken into account, represents at least 90% of the total mass of the structure;
- all modes with effective modal masses greater than 5% of the total mass are intended to be taken into account.

Note: The effective modal mass m_k, corresponding to a mode k, is determined so that the base shear force F_{bk}, acting in the direction of application of the seismic action, may be expressed as $F_{bk}=S_d(T_k).m_k$. It can be shown that the sum of the effective modal masses (for all modes and a given direction) is equal to the mass of the structure.

Whenever the previous two requirements cannot be satisfied (e.g. in buildings with significant contribution from torsional modes), then the minimum number k of modes to be taken into account in a spatial analysis should be satisfied by the two both following conditions:

$$k \geq 3\sqrt{n} \quad \text{and} \quad T_k \leq 0.20 \ sec$$

where: k number of modes taken into account;
n number of stories above the foundation or the top of a rigid basement;
T_k vibration period of mode k.

(2) Combination of modal responses

The elastic response in two vibration modes, mode i and j (including both translational and torsion modes), can be taken as independent of each other if their T_j and T_i periods satisfy the following condition: $T_j \leq 0.9T_i$.

Thus, the maximum value of a seismic action effect, E_E, assessed with all the relevant modal responses (see previous n°.c.(1)) and considered independent from each other, may be taken equal to the square root of the sum of squares of the modal responses (SRSS rule), as follows:

$$E_E = \sqrt{\sum_N E_{Ei}^2}$$

where: E_E seismic action being considered (force, displacement, etc);
E_{Ei} peak value of the seismic action effect due to vibration mode i.

Note: If the response in two vibration modes i and j can not be taken as independent of each other, then more accurate procedures for the combination of modal maximum responses shall be used, such as the Complete Quadratic Combination (CQC rule).

(3) Accidental torsional effects

If a spatial model is used in this method then the accidental torsional effects referred in item 3.b of this article may be determined as the envelope of the effects resulting from the application of static loadings, consisting of sets of torsional moments M_{ai} about the vertical axis of each storey I, defined as follows:

$$M_{ai} = e_{ai}\, F_i$$

where:

- M_{ai} torsional moment at storey i about is vertical axis;
- e_{ai} accidental eccentricity of storey mass i in accordance with point 3b of this Article and for all relevant directions;
- F_i horizontal force acting on storey i, as derived in n^o.3.c.(3) for all relevant directions.

The effects of the above mentioned loads should be taken into account with positive and negative signs (the same sign for all storey), such that the most unfavorable result is produced for the seismic action effect of interest.

Whenever two planar models are used for the analysis in this method, then the torsional effects may be accounted by applying the same rules as indicated for the same situation in the simplified lateral force method of static analysis to the action effects computed according with the combination of modal responses.

e. Combination of the effects of the components of the seismic action

The horizontal seismic action is described by two orthogonal components considered independent and acting simultaneously on the structure.

The methods exposed in this code (simplified lateral force method of static analysis and modal response spectrum dynamic analysis) have for base the estimation of the peak values of seismic action effects during the response to a single component. The action effects are accounted (can be estimated) by combining the two horizontal components of the seismic action (E_x and E_y) by the square root of the sum of the squared values of the action effect due to each horizontal component (SRSS combination).

Annex 4 – Seismic Action

Appendix A – Building structural regularity – Recommended criteria

1. Criteria for regularity in plan

The plan regularity of a structure governs mainly the structural model choice. In fact a structure regular in plan responds to the horizontal components of the seismic action along each main structural direction in an uncoupled way, so enabling to be analyzed, in each main structural direction, using independent planar models.

For a building to be classified as being regular in plan it should comply with all the following conditions.

a. The distribution in plan of the lateral stiffness and mass shall be approximately symmetrical, related to the two orthogonal horizontal axes.

b. The outline of the building vertical structure, in plan, should have a compact configuration, delimited by convex polygonal lines in each floor level. Any single re-entrant corner or recess of the outline of the structure in plan should not leave an area between it and the convex polygonal line enveloping it which is more than 5% of the inside the outline.

   Notes: As an example, if in a structure with a rectangular exterior outline four single reentrant corners exist, close to the rectangular vertices, with a recess of 25% in one direction and 20% in the other, in each reentrant corner, then this structure satisfy this condition of regularity in plan.

   It should be noted that the exterior outline corresponds only to the vertical structural elements of the building, and not including the floors with their balconies or cantilevers. However the occurrence of interior floor openings should respect, in terms of localization and dimensions, the regularity requirements.

c. The in-plan stiffness of the floors (horizontal diaphragms) should be large enough in comparison with lateral stiffness of the vertical structural elements, in a way that the in-plan floor deformation is negligible when compared with the interstorey drifts. If so, the floor deformation will not affect significantly the horizontal distribution of horizontal seismic forces on vertical structural elements.

   Note: Usually this condition no need to be checked by calculation. A reinforced concrete slab with 70mm thickness could be considered as a rigid horizontal diaphragm for not excessive spans and not including large openings, especially in the vicinity of the main vertical structural elements.

d. The aspect ratio of the floor plan, L_{max}/L_{min} , where L_{max} and L_{min} are respectively the larger and the shorter in-plan dimensions of the floor, measured in any two orthogonal directions, should not be higher than 4.

   Note: This limitation is to prevent situations in which, despite the in-plane rigidity of the floors, its deformation due to the seismic action as a deep beam on elastic supports affects the distribution of seismic shears among the vertical structural elements.

e. At each level and for each direction of analysis, *x* and *y*, of approximately symmetrical as required in the previous condition *(a)*, the *static* *eccentricity*, e_0, and the *torsional-radius*, r, shall be in accordance with the two following conditions, which are expressed for the analysis direction y:

$$e_{0x} \leq 0.30 r_x \quad \text{and} \quad r_x \geq l_s$$

where:

e_{0x}=*'static eccentricity'* or the distance between the *center of stiffness* and the *center of mass* of the floor, measured along the x direction (normal to the direction of analysis)

r_x =*'torsional-radius'* or the square root of the ratio of the torsional stiffness to the lateral stiffness in the y direction

l_s = *'radius of gyration'* of the floor mass in plan or the square root of the ratio of the polar moment of inertia of the floor mass in plan, with respect to the center of mass of the floor, to the floor mass. If the mass is distributed uniformly over a rectangular floor with l and b dimensions (including the floor area outside of the outline of the vertical elements of the structural system), then the *radius of gyration* $l_s = \sqrt{[(l^2 + b^2)/12]}$.

Note: This condition aims to ensure that the fundamental period of the translational mode in each one of the two horizontal directions, x and y, is not smaller than the lower torsional mode about the vertical axis z. In this manner the translational responses are privileged against the contribution of excessive torsional responses, and so avoiding coupling situations, which are uncontrollable by design and could be potentially very dangerous.

f. In single storey buildings the *center of stiffness* is the center of the lateral stiffness of all the primary seismic elements, and the *torsional-radius, r,* is the square root of the ratio of the global torsional stiffness, with respect to the center of lateral stiffness, to the global lateral stiffness, in one direction, taking into account all the primary seismic members in this direction.

The primary seismic elements should be here understood as the vertical structural elements, not including beams and slabs. Then those parameters *center of stiffness* and *torsional-radius* could be determined on the basis of the moments of inertia of the cross-sections of the vertical elements, as follows:

- center of lateral stiffness:

$$x_{cs} = \Sigma(xEI_y)/\Sigma(EI_y) \quad \text{e} \quad y_{cs} = \Sigma(yEI_x)/\Sigma(EI_x)$$

- torsional-radius:

$$r_x = \sqrt{[\Sigma(x^2EI_y + y^2EI_x)/\Sigma(EI_y)]} \quad \text{e} \quad r_y = \sqrt{[\Sigma(x^2EI_y + y^2EI_x)/\Sigma(EI_x)]}$$

g. In multi-storey buildings only approximate definitions of the *center of stiffness* and the *torsional-radius* are possible. A simplified definition, for the classification of structural regularity in plan and for the approximate analysis of torsional effects is possible, if the two following conditions are satisfied:

- all the lateral load resisting systems (cores, structural walls or frames) are continuous along the height of the building, from the foundation to their tops;
- the deflected shapes of the individual systems under horizontal loads are not very different. This condition may be assumed as satisfied in frame alone systems (deformation as a shear-beam) or in wall alone systems (deformation as vertical cantilevers), but normally, is not satisfied in dual systems.

This means that, in only frame moment systems or shear-wall systems the determination of the *center of stiffness* and *torsional-radius,* in each storey, may be determined using the above formulation presented in (f) for single storey buildings. However, if the shear deformations, in addition to the flexural ones, are also significant, then an equivalent rigidity (or moment of inertia) of the section should be used in the above formulation.

2. Criteria for regularity in elevation

For a building to be classified as being regular in elevation, it should comply with all the following four conditions:

a. All the lateral load resisting systems (cores, structural walls or frames) shall be continuous along the height of the building or, if setbacks at different heights exist, the following item d. should be followed.

b. Both the lateral stiffness and the mass of the individual storeys shall remain constant or decrease gradually, without abrupt changes, from the base to the top.

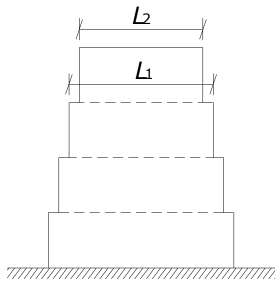
c. In framed buildings the ratio of the actual storey resistance to the resistance required by the analysis should not vary significantly between adjacent storeys.

d. When setbacks are present, the following additional conditions apply:

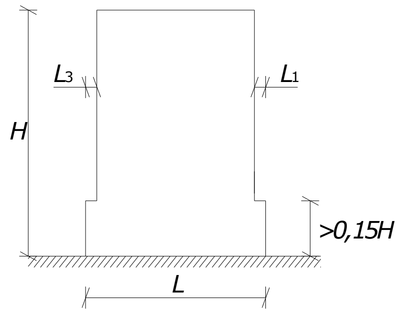
(1) for gradual setbacks preserving axial symmetry, the setback at any floor shall not be greater than 20% of the previews plan dimension in direction of the setback (see Figure IV.A.1.a and IV.A.1.b);

(2) for single setback within the lower 15% of the total height of the main structural system, the setback shall be not greater than 50% of the previous plan dimension (see Figure IV.A.1.c). In this case the structure of the base zone within the verticability projected perimeter of the upper storeys should be designed to resist at least 75% of the horizontal shear forces that would develop in that zone in similar building without the horizontal shear forces that develop in that zone in similar building without the base enlargement;

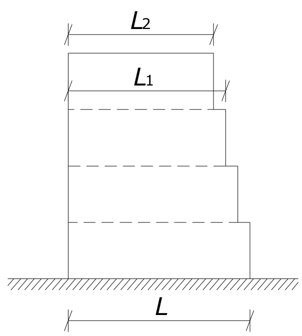
(3) if the setbacks do not preserve symmetry, in each face the sum of the setbacks at all storeys shall not be greater than 30% of the plan dimension at the ground floor above the foundation or above the top of a rigid basement, and the individual setbacks shall be not greater than 10 % of the previews plan dimension (see Figure IV.A.1.d).



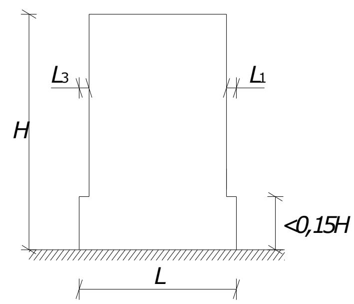
(a) $(L_1-L_2)/L_1 \leq 0.20$



(b) $(L_3-L_2)/L \leq 0.20$



(c) $(L_3+L_1)/L \leq 0.50$



(d) $(L-L_2)/L_1 \leq 0.30$ and
$(L_1-L_2)/L_1 \leq 0.10$

Figure IV.A.1 – Criteria for regularity of buildings with setbacks in elevation

Appendix B – <u>Fundamental vibration period on building structures</u>

For a structure, depending on the nature of its lateral force-resisting structural system, same empirical formulas are allowed to be used to estimate the fundamental vibration period for each one of the considered directions. However, a more direct and dynamical methods could be used, such as the Rayleigh method.

Empirical formulas

(a) Method 1:

According to the type of building structure, following expressions are suggested:

Frame structures $T_1 = n/12$

Dual frame-shear wall structures $T_1 = n/16$

Shear wall structures $T_1 = n/(6b)$

where:
- T_1 structure fundamental vibration period in the respective horizontal direction of interest, in s ;
- n number of floors above ground level;
- b dimension of the building in plant in the direction considered.

(b) Method 2:

For buildings not exceeding 40m in height: $T_1 = C_t . H^{3/4}$

where:
- T_1 structure fundamental vibration period in the respective horizontal direction of interest, in s ;
- H total height of the building, in m;
- C_t is defined according to the following table:

C_t	Type of structures
0.085	steel frame structures
0.075	reinforced concrete frames structures and steel frames structures with diagonal bracings
0.050	all other types of structures
(see calculation note)	concrete or masonry shear walls structures

Calculation note: The value of C_t for this type of structures may be taken as follows: $C_t = 0{,}075/\sqrt{A_c}$

where: $A_c = \sum \left[A_i . \left(0{,}2 + \left(l_{wi} / H\right)\right)^2 \right]$

- A_c total effective area of the shear walls in the first storey of the building (m^2);
- A_i the effective cross-section area of shear wall i in the first storey of the building (m^2);
- l_{wi} the length of the shear wall i in the first storey in the direction of the applied forces with the restriction that l_{wi}/H should not exceed 0.9.