Volume 1, Number 2 (November 2005) p. 145-165 ISSN 1809-1121
Revista IBRACON de Estruturas
IBRACON Structural Journal

The use of γ_z parameter to evaluate second order effects on reinforced concrete buildings

Utilização do parâmetro γ_z para estimar esforços de segunda ordem em edifícios de concreto armado



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Abstract

This paper deals with global second order effects in concrete buildings, discussing simplified procedures for considering geometric non-linearity for structural analysis. A detailed study on the γ_z parameter is carried out showing that increasing the first order internal forces with that parameter is possible to assess the second order effects. Finally, advantages and limitations of its use are clearly established.

Keywords: Reinforced concrete; Tall buildings; Global instability.

Resumo

Neste trabalho são abordados os efeitos globais de segunda ordem em estruturas de contraventamento de edifícios de concreto armado, sendo discutidos os resultados obtidos por um procedimento simplificado para a consideração da não-linearidade geométrica (NLG) na análise de edifícios. Um estudo detalhado do parâmetro γz, proposto como um majorador dos esforços em primeira ordem para a obtenção dos esforços finais em segunda ordem, é apresentado. Desse modo são estabelecidas de forma mais clara e objetiva as vantagens e as limitações deste procedimento.

Palavras-chave: Concreto armado, Edifícios altos; Instabilidade Global.

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1 Introduction

When analyzing tall building structures, the designer has to deal with the problem of global stability, because the structure is simultaneously subjected by vertical and horizontal loadings. In fact, the induced additional actions due to horizontal displacements of the structure may cause an increase in internal forces that can lead it to collapsing.

It is important to mention that the horizontal displacements are present (to a large or small degree) in almost all constructions, caused by several sources, such as wind action, geometric or loading asymmetries, building imperfections, etc.

This kind of analysis, in which the equilibrium of the structure is based on its deformed position, is called geometric nonlinear analysis (GNL) or second order analysis.

The usual design process requires assessing the degree of mobility of the structure in a simple way, without performing a complete second order analysis. Safe and easy practical criteria have been researched to fulfill this goal in order to classify the structures, based on the mobility degree, in a not complex way.

CEB-FIP Code modèle 77[1] incorporated the α parameter, idealized by BECK&KÖNIG [2] for a cantilever column with the vertical loading uniformly distributed along the height. FRANCO [3] shows that one can use this parameter for multi-storey buildings, using the concepts of "equivalent stiffness" and "shape parameter of the elastic curve."

The CEB-FIP Model Code 90 [4] has abandoned the α parameter. In order to fill in this gap, FRANCO & VASCONCELOS[5] have proposed the γ_z parameter as a new instability parameter. This new parameter has the additional advantage of being a forecast of the final second order internal efforts, and not only a measure of the mobility degree of the structure.

Finally, it is worthy of note that the instability parameters are easily implemented because of their simplicity, becoming useful design tools to evaluate the second order effects in the structure.

2 The γ_z parameter

This parameter was proposed by FRANCO & VASCONCELOS [5] as a magnification coefficient of the first order internal forces to obtain the final ones, including the second order effects.

Starting with a linear analysis for the horizontal loading, and the assessment of the first order global moment M_1 at the base of the structure, the vertical forces multiplied by the horizontal displacements of each floor cause increments of moments ΔM_2. These increments provide the appearance of new displacements and the process repeats successively during several iteration stages until the increments become insignificant, in the case of stable structures. The sum of the various moment increments equals the final second order moment:

$$M_2 = M_1 + \Delta M_2 + \Delta M_3 + \ldots + \Delta M_j \qquad (1)$$

where *j* is the number of iterations, taking M_1 for the first iteration.

The CEB-FIP Manual of Buckling and Instability [6] suggests that moments M_1 , ΔM_2 , ΔM_3 , ΔM_j constitute a decreasing geometrical series of ratio $r \leq 1$.

Then:

$$r = \frac{\Delta M_2}{M_1} = \frac{\Delta M_3}{\Delta M_2} = \ldots = \frac{\Delta M_j}{\Delta M_{j-1}} \qquad (2)$$

With $\Delta Mj = r \,.\, \Delta Mj\text{-}1$, equation (1) becomes, then:

$$M_2 = (1 + r + r^2 + r^3 + \ldots + r^{j-1})\, M_1 \qquad (3)$$

When j tends towards infinite, the sum of the progression terms of equation (3) is:

$$\lim_{j \to \infty} \left(1 + r + r^2 + r^3 + \ldots + r^{j-1}\right) M_1 = \frac{1}{1-r} M_1 \qquad (4)$$

The ratio $r = \Delta Mj / \Delta Mj\text{-}1$ can be written as $r = \Delta M_2 / \Delta M_1 = \Delta M / M_1$. Considering the linear analysis as j =1, $\Delta M_2 = \Delta M$ and $\Delta M_1 = M_1$. Taking the Ultimate Limit State (ULS) values for the involved variables, equation (4) reads:

$$M_{2d} \cong \frac{1}{1 - \dfrac{\Delta M_d}{M_{1d}}} M_{1d} \qquad (5)$$

Considering γ_z parameter as the factor that magnifies moment M_{1d}:

$$\gamma_z = \frac{1}{1 - \dfrac{\Delta M_d}{M_{1d}}} \,. \qquad (6)$$

Then, the second order effects of the structure can be evaluated, in a simplified way, from the results of a first order analysis.

According to the CEB-FIP Model Code 90 [4] a building can be considered as non-sway if the second order effects, induced by the horizontal displacements of the structure, cause increases lower than 10% of the relevant first order bending moments. This criterion is known as the immobility condition of the joints. Therefore, if $\gamma_z \leq 1{,}1$ the structure can be classified as non-sway.

3 Use of the γ_z parameter for estimating second order effects

In order to study the γ_z, parameter in a more detailed way and its real capability of approaching the second order effects, several buildings are analyzed in the first and second order to assess the increases of the internal forces of the structural elements. The increases are compared with those predicted by the simplified procedure, in which the final second order internal forces are obtained by the magnification of the first order forces with the γ_z parameter.

The methodology to be used, in a systematic way, consists of the following steps:

a) Execution of a first order analysis of the structure subjected only to the horizontal loading, taking into account the physical non-linearity (PNL) in a simplified way, by a reduction of the bending stiffness of the structural elements;

b) Computation of the first order internal forces in all structural elements;

c) Assessment of the γ_z parameter values corresponding to the two main horizontal directions of the building, called x and y;

d) Analysis of the structure in the second order theory, including horizontal and vertical loadings, considering GNL by incremental changes of the stiffness matrices and PNL in a simplified way;

e) Computation of the second order internal forces in all structural elements;

f) Comparison of the second order results to the first order ones magnified by the γ_z parameter, for the whole structure and for adjacent zones along the height of the building.

It is worth mentioning that the afore-mentioned comparisons involve the average results, since the dispersions of values are small, as can be verified in PINTO[11].

The proposed analyses were performed by the computational system LASER, developed by RAMALHO[7] and upgraded by CORRÊA[8] to include GNL. This system can analyze 3D structures, with frame elements, subjected to nodal forces.

4 Basic concepts and adopted simplifications

This section presents the main used concepts and the adopted simplifications for the analysis.

4.1 Physical non-linearity of the material

PNL will be considered in a simplified way, taking into account the suitable properties suggested by FRANCO [9]:

a) Columns: I=0,8 Ig

b) Beams (reinforcement in both faces): I=0,5 Ig

Beams (reinforcement in one face): I=0,4 Ig

Where I is the reduced moment of inertia and Ig is the gross moment of inertia of the cross-sectional area.

4.2 Applied factors to the vertical and horizontal loadings

For the applied loading in the structure, different safety factors are proposed according to NBR 8681 [10]. This code prescribes that, when considering the geometric non-linearity, the γ_f factor can be factored in the partial coefficients γ_{f1}, γ_{f2} e γ_{f3}:

$$S_d = \gamma_{f3} \cdot S(\gamma_{f1} \cdot \gamma_{f2} \cdot F_k) \qquad (7)$$

where S_d is the design value and F_k the characteristic value of the actions.

The factor $\gamma_{f2} = \psi_0$ is the combination factor, defined in NBR 8681[10], whose prescribed values are:

a) ψ_0= 0.4 for general cases

b) ψ_0= 0.7 for high concentrations of people

c) ψ_0= 0.8 for bookstores, garages, etc.

The γ_{f1} factor accounts for the variability of the actions and the γ_{f3} factor considers potential mistakes in the evaluation of those actions.

In cases where a GNL analysis is performed, NBR 8681 [10] prescribes that γ_{f3} should not be taken smaller than 1.10. In the present work γ_{f3} = 1.15.

Based on FRANCO&VASCONCELOS [5], it is reasonable to admit that 80% of the total load is dead load and 20% is live load. Hence, the vertical loading and the horizontal forces can be factored this way:

Permanent loading (ψ_0 = 1):

$\gamma_g = 1.3$

$\gamma_f = \gamma_g = 1.3 = \gamma_{f1} \cdot \gamma_{f3} = \gamma_{f1} \cdot 1.15$

$\gamma_{g1} = 1.130$

Variable loading:

$\gamma_q = 1.4$

$\gamma_f = \gamma_q = 1.4 \cdot \psi_0 = \gamma_{f1} \cdot \psi_0 \cdot \gamma_{f3} = \gamma_{f1} \cdot \psi_0 \cdot 1.15$

$\gamma_{q1} = 1.217$

$$\text{Then}, \gamma_f = \frac{\gamma_{g1} \cdot g + \gamma_{q1} \cdot q \cdot \psi_0}{g + q} = 0.8 \cdot \gamma_{g1} + 0.2 \cdot \gamma_{q1} \cdot \psi_0$$

$$\gamma_{fv} = 0.904 + 0.243 \cdot \psi_0 = \begin{cases} 1.001 \rightarrow \psi_0 = 0.4 \\ 1.074 \rightarrow \psi_0 = 0.7 \\ 1.098 \rightarrow \psi_0 = 0.8 \end{cases}$$

Considering the horizontal loading as the main live load (ψ_0 = 1), one can adopt:

$$\gamma_{fh} = 1.4 = \gamma_{f1} \cdot \gamma_{f3} = \gamma_{f1} \cdot 1.15 \therefore \gamma_{f1} = 1.217$$

In short, the following values are used for the building analysis in this paper:

Vertical loading:

γ_{fv} = 1.00 (general cases)

Horizontal loading:

γ_{fh} = 1.22

It is important to point out that, after the analysis, one should multiply the obtained results by γ_{f3} = 1.15, as established by Equation (7).

5 Evaluation of the use of the γ_z parameter to evaluate second order effects

In this section the obtained results for 25 reinforced concrete buildings, considering two different horizontal directions, are presented according to PINTO [11]. Therefore, there are 50 different structures when considering the building bracing systems. The obtained results can be used to evaluate how the γ_z parameter estimates the second order increases in the internal forces caused by the horizontal loading.

All of the buildings are real cases designed by different structural engineers. Hence, this set of samples can be considered representative of the bracing structures usually designed in a Brazilian technical environment.

Table 1 presents the buildings analyzed in this paper showing their names, wind direction, number of floors, values of γ_z parameter and the city where they should be built. It is important to highlight that the structures are sorted from the smaller to the bigger value of the γ_z parameter.

Some of the structural systems correspond to the initial phase of design, which explains the γ_z parameter values beyond recommended limits. Therefore, a wide range of γ_z parameters values to evaluate the coefficient γ_z can be obtained.

Comparisons were made for the whole structure and also dividing the height into 5 adjacent zones. These zones were established so that the first zone was between the foundation level and the first floor of the building. The other zones were distributed as uniformly as possible along the height of the structure.

The results are presented in graphs that show the differences between the rigorous procedure for GNL and the estimated values obtained with the γ_z parameter. In the vertical axis the γ_z parameter value instead of the number of the structure is always shown. This option highlights the percentage differences related to the γ_z parameter, which is the main goal of this work.

According to the adopted scheme, when the differences are positive, i.e., when the bar appears on the right side of the zero line, the value estimated by the γ_z parameter is unsafe. On the other hand, if the difference is negative, i.e., when the bar is on the left side of the zero line, the γ_z parameter estimation is safe.

5.1 Axial forces in columns

The average increases of second order effects for axial forces in columns are presented in Figure 1.

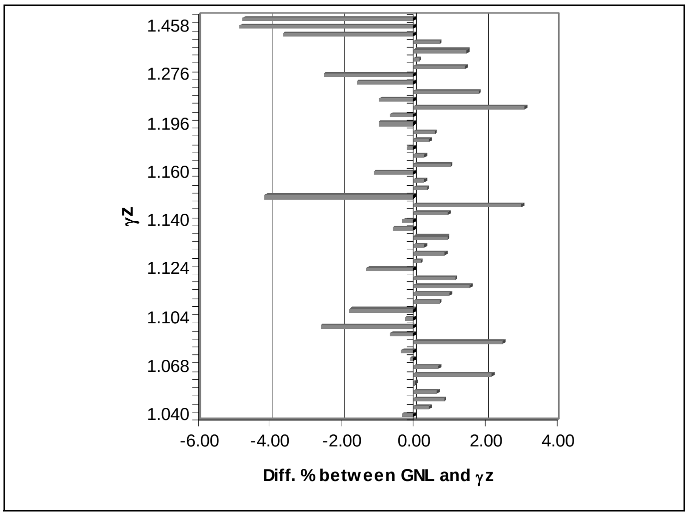

Figure 1 - Diff. % between increases with GNL and γz - Axial forces in columns – Global.

Those values were evaluated considering only the horizontal loads on the building. In other words, they were obtained by deducting from the final values of the axial forces the effects due to the vertical loads. Of course, in this case, the principle of superposition was applied.

In general, regarding the obtained results, the γ_z parameter provides a good evaluation of the second order increases for axial forces in columns. The differences are smaller than 5% and the γ_z parameter overestimates the second order effects for the biggest values.

Table 1 - Analyzed Structures.

Nº	Estrutura	Dir	Nº Pav	γz	Localização
1	Saint Regis	y	16	1,040	Campinas-SP
2	Saint Regis	x	16	1,043	Campinas-SP
3	Padova-Luca	y	16	1,047	Santos-SP
4	Conde do Pinhal	y	14	1,059	São Carlos-SP
5	Spazio Uno	x	17	1,060	Rib. Preto-SP
6	Córsega	y	18	1,065	São Paulo-SP
7	Andaluzia	y	20	1,068	Sto André-SP
8	Stradus	y	12	1,071	Brasília-DF
9	Maison Bougainville	y	20	1,076	Sto André-SP
10	Córsega	x	18	1,095	São Paulo-SP
11	Corinto	x	18	1,099	São Paulo-SP
12	Andaluzia	x	20	1,104	Sto André-SP
13	Torre Perdizes	y	30	1,104	São Paulo-SP
14	Porto Bello	y	11	1,111	Manaus-AM
15	Maison Etoile	x	21	1,113	São Paulo-SP
16	Ville Dijon	y	15	1,113	Taubaté-SP
17	Maison Etoile	y	21	1,116	São Paulo-SP
18	Ville Florence	x	16	1,122	Jundiaí-SP
19	Ville Florence	y	16	1,124	Jundiaí-SP
20	Maison Bougainville	x	20	1,128	Sto André-SP
21	Porto Bello	x	11	1,129	Manaus-AM
22	Ville Dijon	x	15	1,130	Taubaté-SP
23	Premium	y	15	1,133	Goiânia-GO
24	Corinto	y	18	1,138	São Paulo-SP
25	Av. Circular	y	14	1,140	Goiânia-GO
26	Torre Perdizes	x	30	1,141	São Paulo-SP
27	Lion Dior	y	19	1,151	Rib. Preto-SP
28	Conde do Pinhal	x	14	1,156	São Carlos-SP
29	Spazio Uno	y	17	1,157	Rib. Preto-SP
30	Cartier Tower	y	18	1,159	Rib. Preto-SP
31	Premium	x	15	1,160	Goiânia-GO
32	Butantã	x	15	1,162	São Paulo-SP
33	Lion Dior	x	19	1,170	Rib. Preto-SP
34	J. F. Guimarães	y	18	1,174	Rib. Preto-SP
35	Padova-Luca	x	16	1,183	Santos-SP
36	Maison Classic	x	15	1,195	Recife-PE
37	Espaço São Paulo II	y	21	1,196	São Paulo-SP
38	Rua Indiana	y	25	1,199	São Paulo-SP
39	Av. Circular	x	14	1,209	Goiânia-GO
40	Top Life	y	20	1,225	Juiz de Fora-MG
41	Butantã	y	15	1,257	São Paulo-SP
42	Espaço São Paulo II	x	21	1,261	São Paulo-SP
43	Top Life	x	20	1,276	Juiz de Fora-MG
44	Cartier Tower	x	18	1,277	Rib. Preto-SP
45	J. F. Guimarães	x	18	1,290	Rib. Preto-SP
46	Maison Classic	y	15	1,298	Recife-PE
47	Condomínio III	y	24	1,389	São Paulo-SP
48	Condomínio III	x	24	1,444	São Paulo-SP
49	Stradus	x	12	1,458	Brasília-DF
50	Rua Indiana	x	25	1,557	São Paulo-SP

5.2 Bending moments in columns

The differences between second order effects for bending moments in columns for the whole structure are presented in Figure 2. For γ_z parameter up to 1.15, the differences are about 2%, sometimes overestimating and sometimes underestimating the second order effects, except for structure 23 (γ_z = 1.133) where the difference is 3.7% and the γ_z parameter underestimates the second order effect.

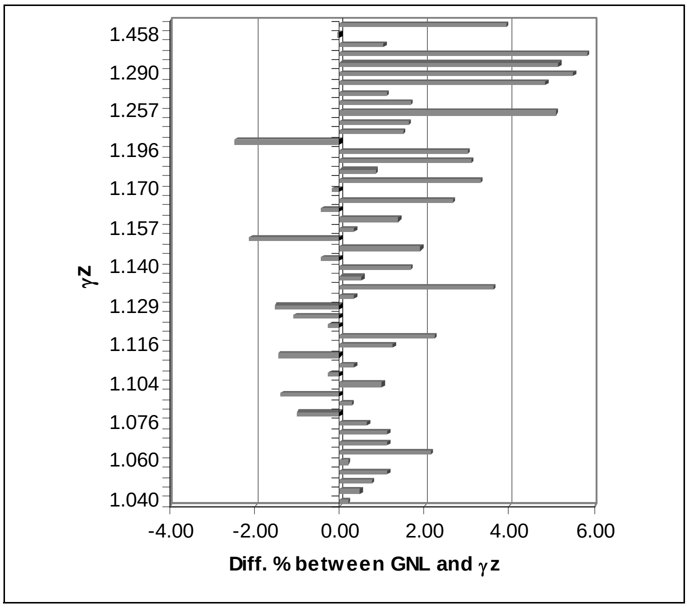

Figure 2 - Diff. % between increases with GNL and γ_z – Bending moments in columns – Global.

For γ_z parameter values between 1.15 and 1.25, differences of about 3% can be seen. Over 1.25 the differences are over 5% and in general the γ_z parameter underestimates the second order effect.

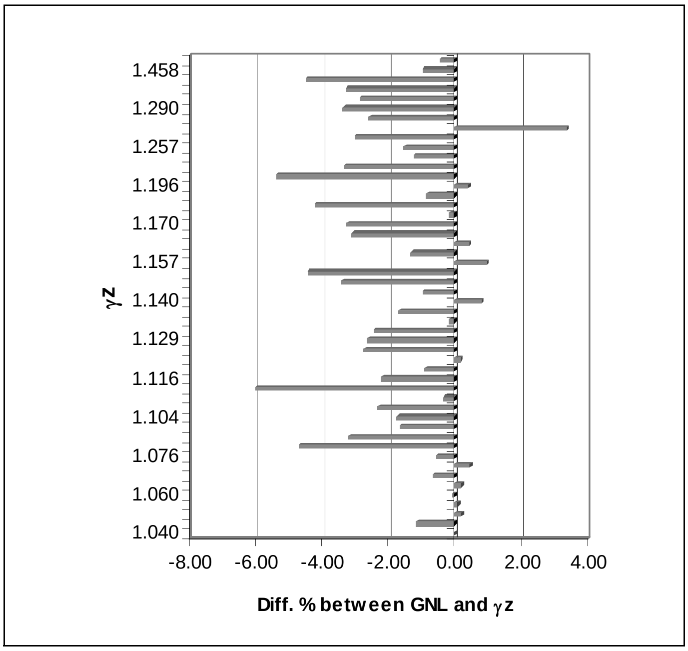

Figure 3 - Diff. % between increases with GNL and γ_z – Bending moments in columns – Zone 1.

Figure 3 shows the obtained results for zone 1. For most of the buildings, the γ_z parameter overestimates the second order effects. For this zone, considering the general trends, the most discrepant result was that of structure 43 (γ_z = 1.276) for which the γ_z parameter underestimates the second order effect in 3.2%.

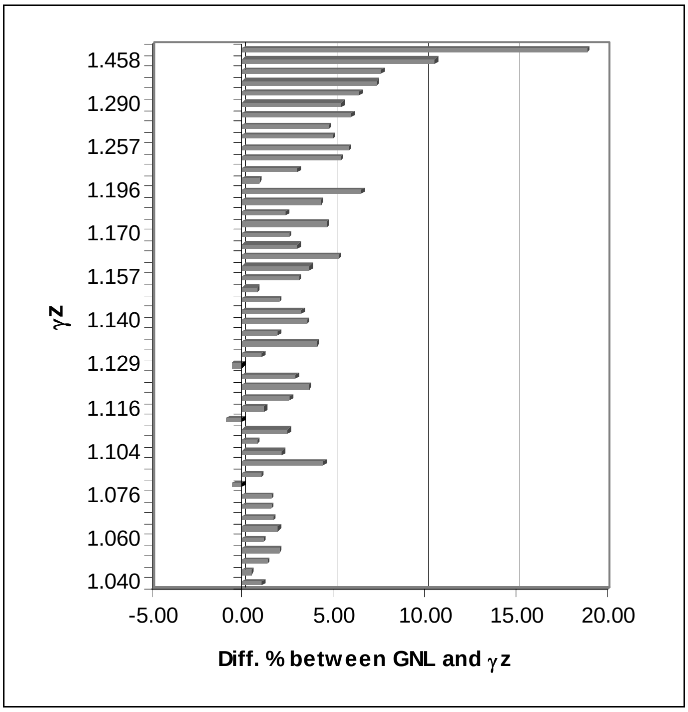

Figure 4 - Diff. % between increases with GNL and γ_z – Bending moments in columns – Zone 2.

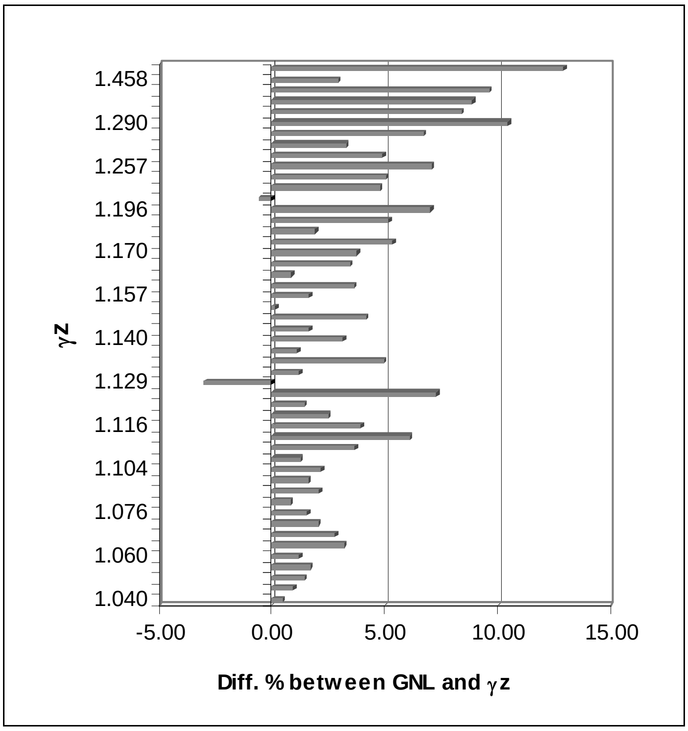

Figure 5 - Diff. % between increases with GNL and γ_z – Bending moments in columns – Zone 3.

Figure 4 shows the obtained results for zone 2. One can observe that the γ_z parameter underestimates the second order effects for most of the structures. In other words the second order effects calculated with GNL is generally bigger than the values estimated by the γ_z parameter and the differences enlarge with the γ_z parameter growth. For γ_z parameter beyond 1.25, the differences are over 6% on the unsafe side of the graph reaching about 19% for structure 50 (γ_z = 1.557).

The differences for zone 3 behave similarly to zone 2. However, for zone 3 the number of structures that present differences over 5% on the unsafe side of the graph is bigger, even for lower values of the γ_z parameter, as can be seen in Figure 5.

For zone 4 the γ_z parameter sometimes overestimates and sometimes underestimates the second order effects. This behavior is almost independent of the γ_z parameter values, as can be shown in Figure 6. For some structures, the γ_z parameter overestimates the second order effects by about 10%. On the other hand, differences on the unsafe side of the graph larger than 10% can be found only for γ_z parameter values over 1.25.

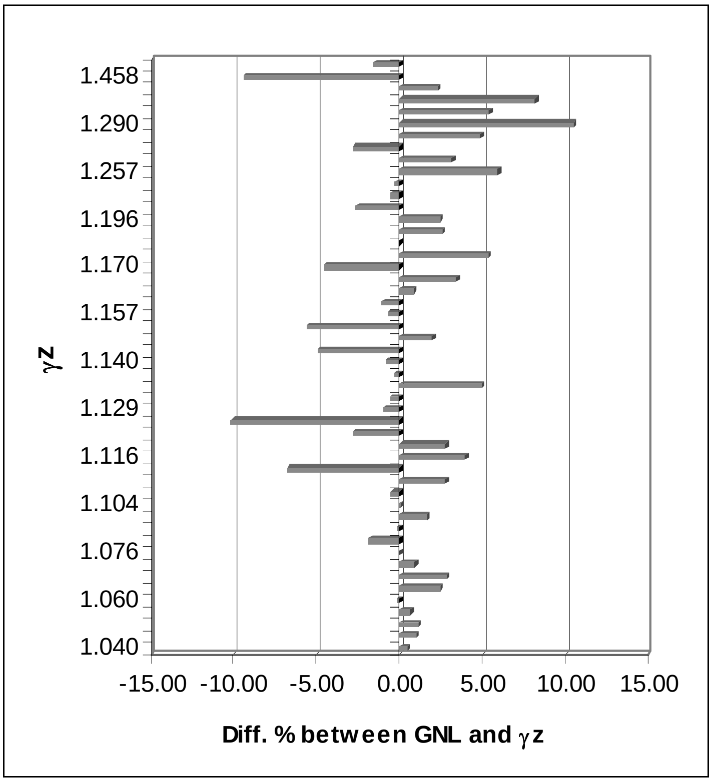

Figure 6 - Diff. % between increases with GNL and γ_z – Bending moments in columns – Zone 4.

Finally, Figure 7 shows that the simplified procedure tends to overestimate results for zone 5, almost independently of the magnitude of the γ_z parameter values.

In general, the bigger the γ_z parameter the bigger the differences between the simplified procedure and the results obtained by the GNL. Generally the γ_z parameter overestimates the second order effect for zones 1 and 5 and underestimates the second order effect for zones 2 and 3. For zone 4, the tendency is not well defined and the obtained results can be either on the safe or on the unsafe side of the graph.

Considering that in any case it is not adequate to have the γ_z parameter over 1.25, it is possible to see that the maximum error for the second order effect considering bending moments in columns would be about 5%. This error seems to be acceptable especially considering the advantages of the simplified method and the fact that this procedure is adequate only for the design of usual structures.

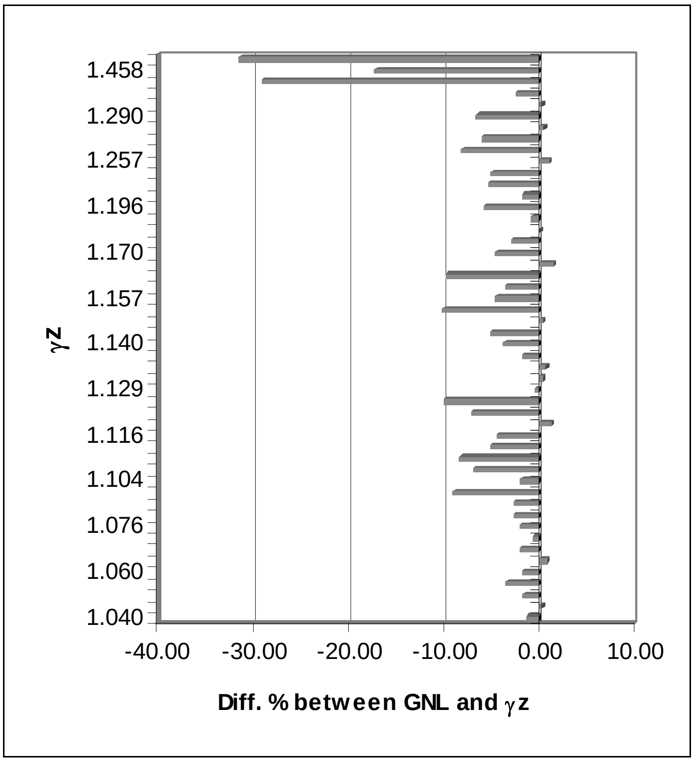

Figure 7 - Diff. % between increases with GNL and γ_z – Bending moments in columns – Zone 5.

5.3 Shear forces in beams

Results in Figure 8, for the structure as a whole, shows that the average increases of the bending moments are similar to the results predicted by the γ_z parameter, even for the highest values. For γ_z below 1.25 the larger difference on the unsafe side is about 2%. For γ_z beyond 1.25, the differences are slightly larger and can reach 3% on the unsafe side.

The analysis of the γ_z behavior along the building's height shows that for zone 1 (Figure 9) the average increases oscillate about 2% from the safe to the unsafe side in most of the structures. The largest differences on the unsafe side occur for γ_z values beyond 1.4. However, structure 31 (γ_z = 1.16), shows a difference of 4% on the unsafe side, which does not match with the general trends for all other structures.

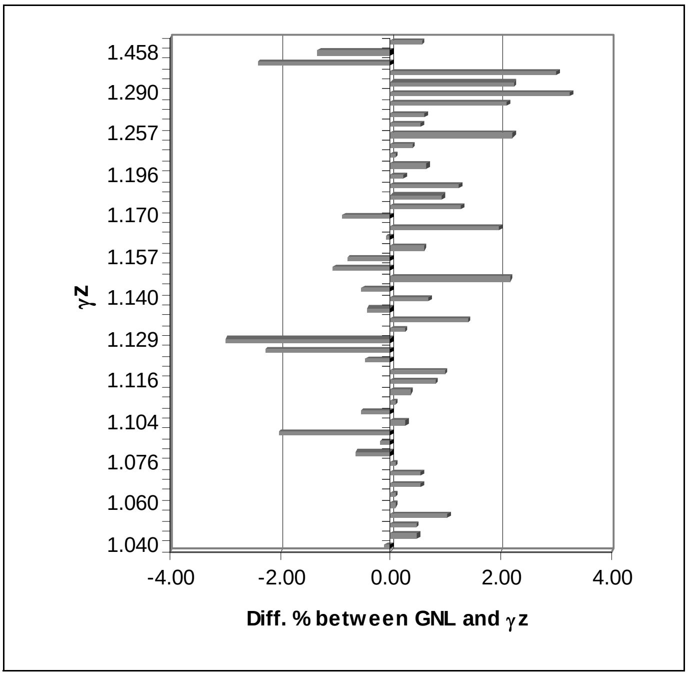

Figure 8 - Diff. % between increases with GNL and γ_z – Shear forces in beams –Global.

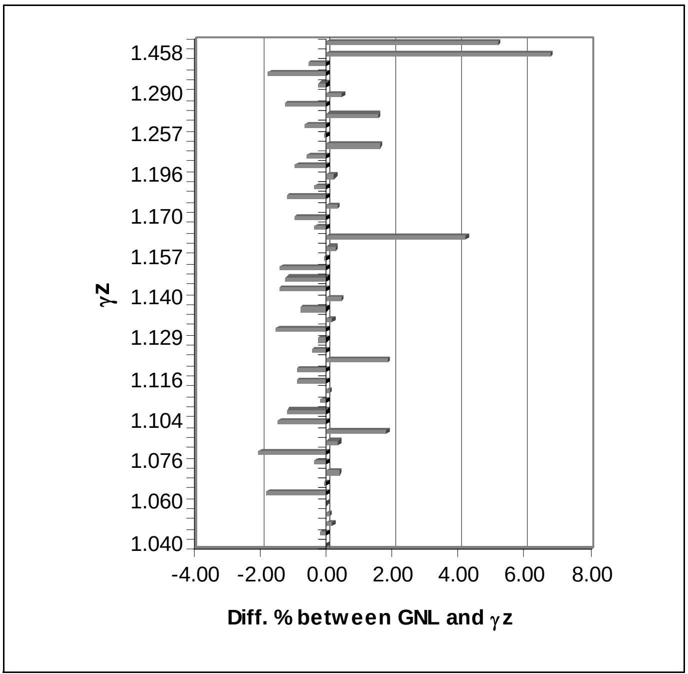

Figure 9 - Diff. % between increases with GNL and γ_z – Shear forces in beams – Zone 1.

Figure 10 shows the obtained results for zone 2, which shows that the γ_z parameter is generally unsafe. All the structures show 2nd order effects larger than those from the simplified procedure, except 6 and 21, whose GNL results are nearly 5% smaller.

Note that the differences between the GNL and the simplified procedure become larger with the increase of the γ_z parameter. However, these differences surpass 5% only for γ_z larger than 1.30.

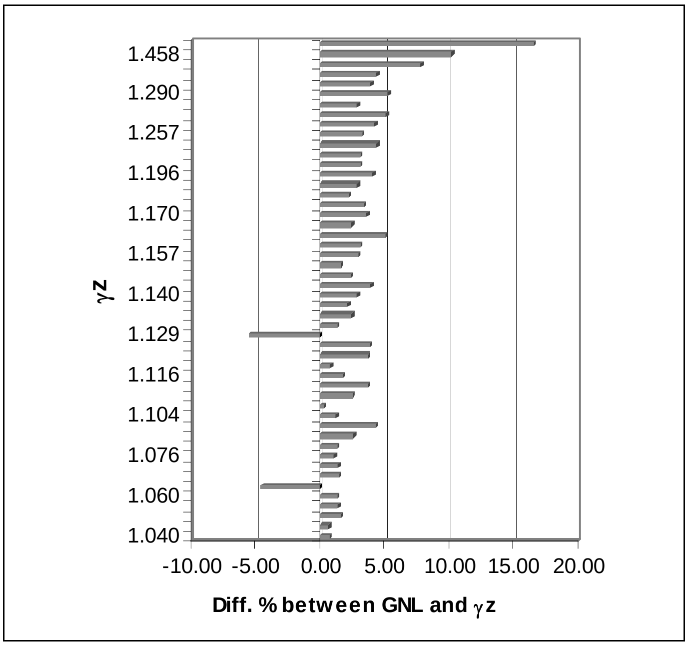

Figure 10 - Diff. % between increases with GNL and γ_z – Shear forces in beams – Zone 2.

Zone 3 (see Figure 11) shows similar results to Zone 2, except for structure 33 (γ_z = 1.170) whose average GNL results are 9.5% larger than those from γ_z parameter.

On the other hand, for zone 4 (Figure 12) the γ_z parameter estimates are safe for most of the structures. The differences on the safe side reach 11% even for low γ_z values, reaching 13% for values larger than 1.30. Differences on the unsafe side are small, always below 4.5%.

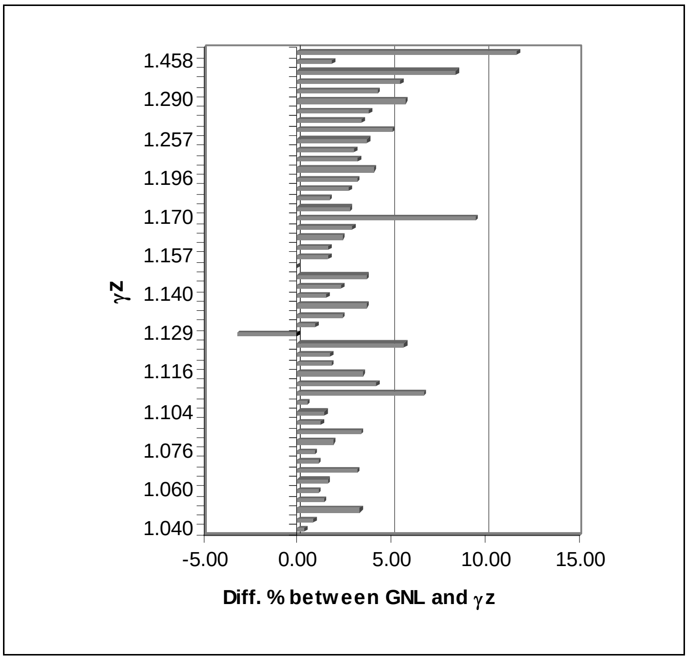

Figure 11 - Diff. % between increases with GNL and γ_z – Shear forces in beams – Zone 3.

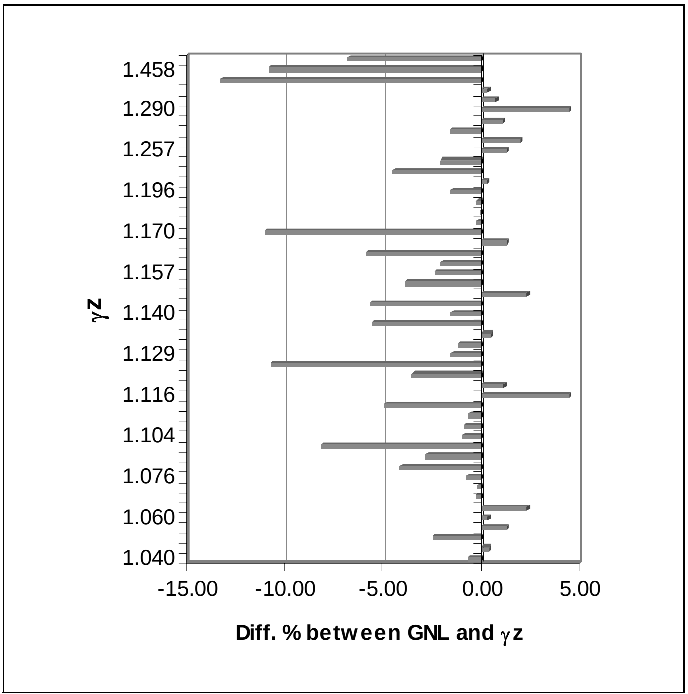

Figure 12 - Diff. % between increases with GNL and γ_z – Shear forces in beams – Zone 4.

Finally, the simplified values for shear forces on beams of the 5th zone are consistently on the safe side. Many GNL values are 5% smaller than the γ_z estimates and the higher differences are close to 30%. However, it is worthy mentioning that the most significant values are on the safe side and that the shear forces in this zone are not very important when considering their absolute values.

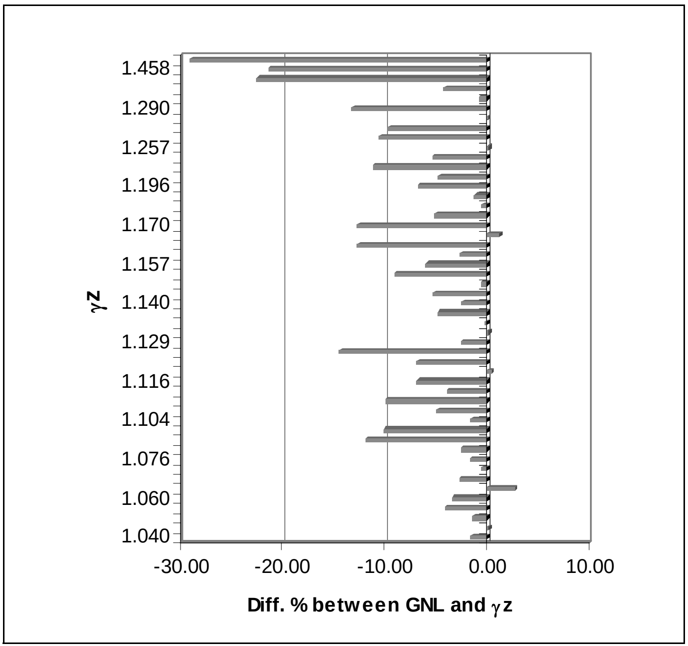

Figure 13 - Diff. % between increases with GNL and γ_z – Shear forces in beams – Zone 5.

In general, the beam shear forces are well estimated by the γ_z parameter. For the most significant regions, Zones 1, 2 and 3, the percentage differences between the simplified procedure and GNL do not go beyond 5% for γ_z values lower than 1.25. Only for zone 4 and especially zone 5 those differences tend to increase, being always on the safe side. Since the first order results in these last zones are small, the overestimates produced by the simplified procedure are not serious drawbacks for design purposes.

5.4 Bending moments in beams

The average increases in the beam's bending moments from the GNL analysis are similar to those estimated by the γ_z parameter. Values on Figure 14 confirm this fact for the whole structure. For γ_z lower than 1.25, the differences (close to 2%) oscillate between the safe and unsafe side, except for the structure 21 (γ_z = 1.129), whose GNL values are nearly 3% lower than the γ_z estimates. On the other hand, differences slightly beyond 3% on the unsafe side appear for γ_z higher than 1.25.

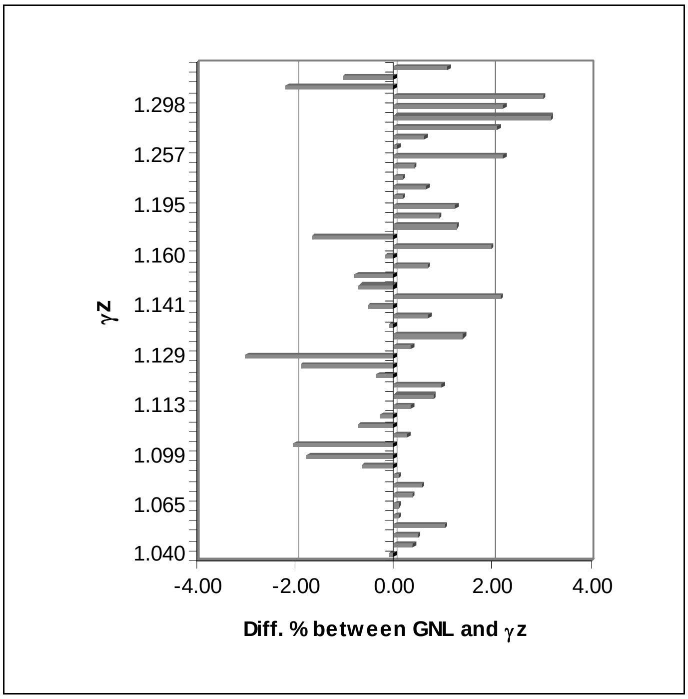

Figure 14 - Dif. % between increases with GNL and γ_z results – Bending moments in beams – Global.

Figure 15 shows results for zone 1, where differences between the GNL and γ_z estimates are lower than 2%, for γ_z below 1.25.

The only exception in zone 1 is that of structure 31 (γ_z = 1.160), whose difference is slightly larger than 4% on the unsafe side.

Zone 2 (Figure 16) shows for most of the examples differences on the unsafe side, but lower than 5% for γ_z parameter below 1.25. Structures 6 (γ_z = 1.065) and 21 (γ_z = 1.129) are exceptions, since the differences are on the safe side. However the differences enlarge significantly for γ_z beyond 1.30, reaching values higher than 15% on the unsafe side.

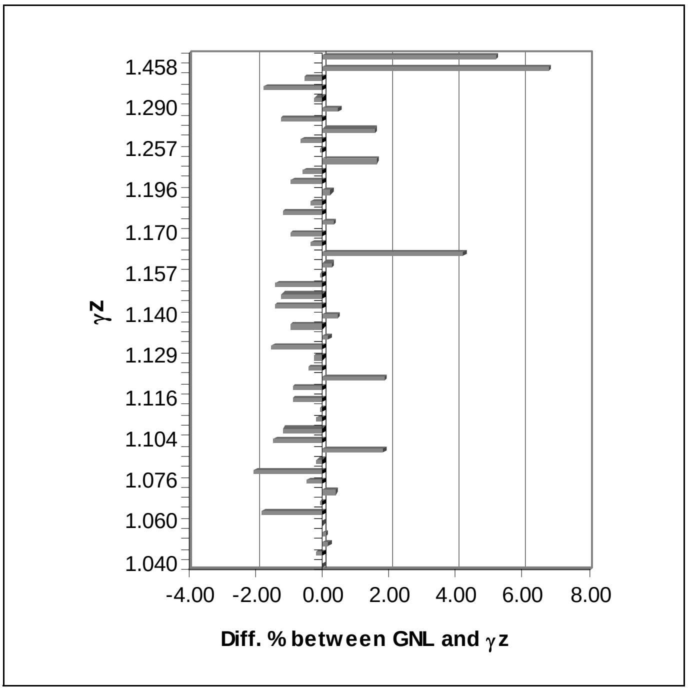

Figure 15 - Dif. % between increases with GNL and γ_z results – Bending moments in beams – Zone 1.

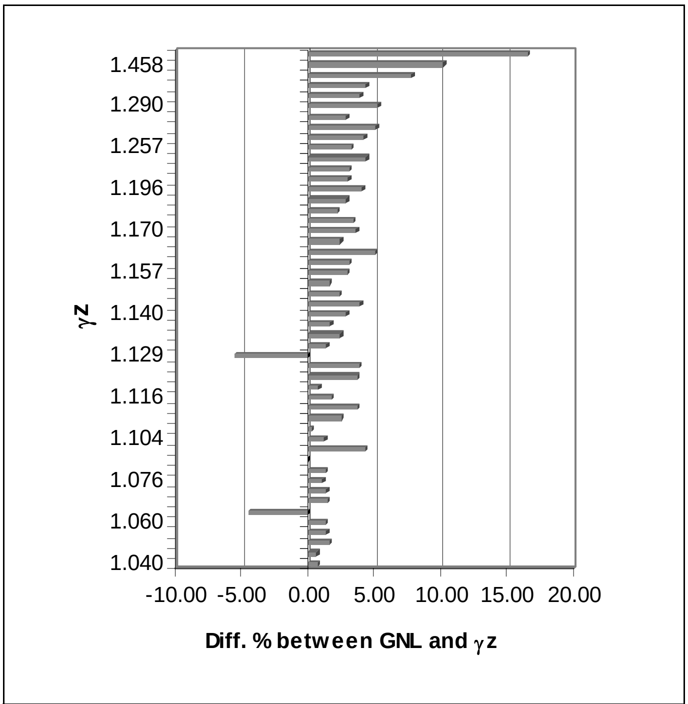

Figure 16 - Dif. % between increases with GNL and γ_z results – Bending moments in beams – Zone 2.

Figure 17 shows the pattern of differences for zone 3. Similarly to zone 2, the occurrence of unsafe results is a general tendency. It has to be enhanced that a higher number of samples show percentage differences larger than 5% on the unsafe side, even for small γ_z values. For instance, structures 20 (γ_z = 1.128) and 33 (γ_z = 1.170) that show differences close to 8% on the unsafe side.

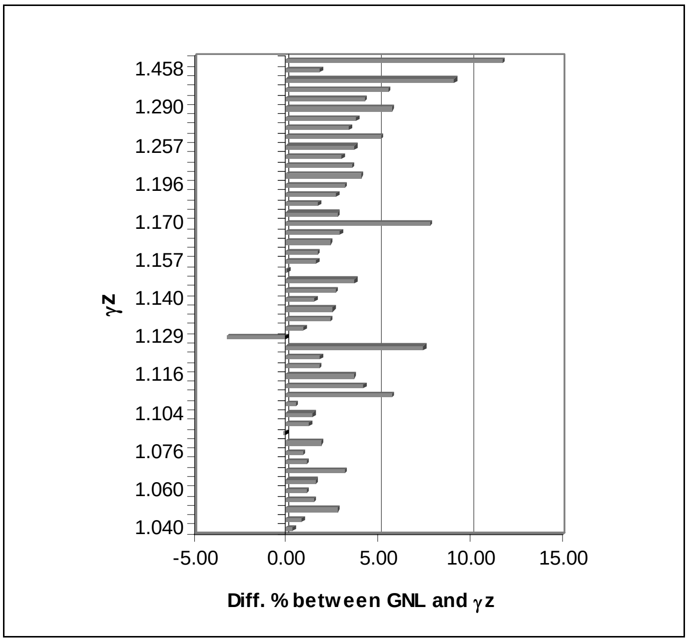

Figure 17 - Dif. % between increases with GNL and γ_z results – Bending moments in beams – Zone 3.

For zone 4 (Figure 18) most of the structures show safe differences between the GNL and the simplified procedure. These differences approach 13% even for relatively low values of the γ_z parameter. All of the unsafe differences are smaller than 4%.

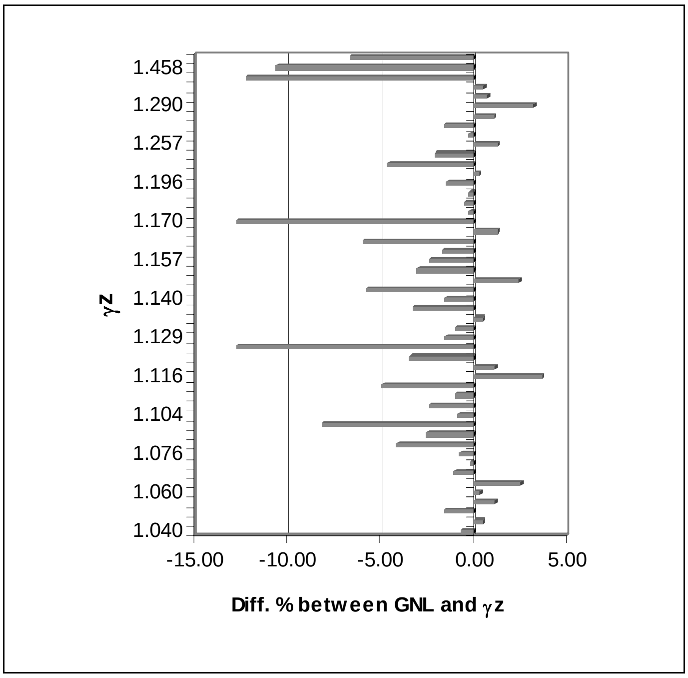

Figure 18 - Dif. % between increases with GNL and γ_z results – Bending moments in beams – Zone 4.

Finally, there is a tendency for producing safe differences between the GNL and the γ_z parameter and some of them are close to 30%. However, as observed before, the absolute values of the bending moments are small and the differences are not significant for design purposes.

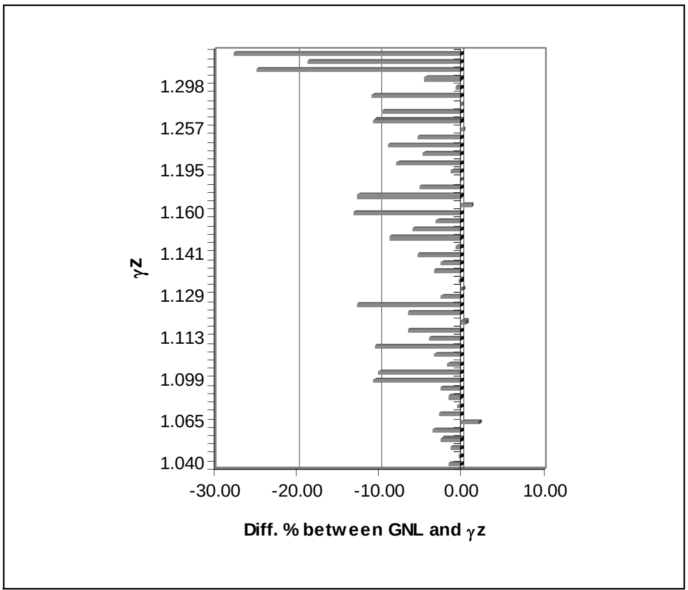

Figure 19 - Dif. % between increases with GNL and γ_z results – Bending moments in beams – Zone 5.

6 Conclusions

The present paper showed results for 50 structures, generated by two different 2nd order analysis procedures: a simplified one, in which the first order internal forces are multiplied by the γ_z parameter and a rigorous one, which takes into account the GNL by means of incremental stiffness matrices.

In both procedures, a reduction of the moment of inertia of cross-sectional areas simulates the NLF in the structural elements. The comparison of the alternative results obtained for the internal forces allows for the evaluation of the accurateness of the simplified procedure for the entire building and for different zones across its height.

Regarding the axial forces in the columns, the GNL results are similar to those predicted by the γ_z parameter for the structure as a whole. The maximum differences are 3%, on the unsafe side, and 5% on the safe side.

The highest differences for the column bending moments, considering the structure as a whole, are smaller than 4% for γ_z below 1.20. For values beyond 1.20, the differences tend to overmatch 5%, with most of the values on the unsafe side. The differences between the GNL and simplified results are on the safe side for column portions near the base. In intermediary portions the differences are unsafe, returning to the safe side near the top. It is worth mentioning that the differences on the unsafe side are consistently lower than 5% for γ_z values below 1.25.

Shear forces and bending moments in the beams show very similar behaviors because of their dependency.

Regarding the structure as a whole, the differences are smaller than 2%, on the unsafe side, for γ_z values below 1.25 and reach 3% beyond that limit.

Considering the behavior along the building height, the differences for internal forces in the beams oscillate from the safe to unsafe side near the base. The maximum difference on the unsafe side is 4% for a γ_z parameter of 1.25. For values beyond 1.30 the differences on the unsafe side reach 7% in this zone.

Regarding the structural elements in the intermediate zones, the γ_z estimates are unsafe, with differences up to 5% for γ_z parameters up to 1.25. For values beyond 1.30, the differences increase and can reach 17%.

Finally, for beam elements near the top, the simplified procedure shows safe results, even with some differences close to 30%. However, it is worth highlighting that the absolute values of the internal forces are low and the obtained differences are not significant in a practical sense.

From all the results, it is clear that the γ_z parameter provides good estimates for the 2nd order effects, for design purposes, under specific limits. The limit 1.20, suggested by FRANCO & VASCONCELOS[5], seems to be a little conservative, and can be extended to 1.25.

The use of the γ_z parameter estimates for values beyond 1.25 should be avoided, since they are on the unsafe side in the intermediate zones, where the internal forces caused by horizontal loading are the highest in the building.

To sum up, under the limit of 1.25, the maximum differences on the unsafe side are close to 5%, which is quite adequate for designing usual structures.

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