0146

NECESSARY STATIC CONDITIONS FOR THE SEISMIC VIABILITY OF REINFORCED CONCRETE OR MASONRY STRUCTURES

Mario M PAPARONI[1]

SUMMARY

Due to a long tradition in the field of reinforced concrete, Shear, Moment and Axial Loads resistances are considered as mutually independent for design purposes. The last two were integrated in the interaction diagrams, but shear continues to be treated as independent, except in a few cases. The Author has developed some algebraic relations, based on the prescriptions of the seismic codes, where one can see that it is not possible, in many cases, to dimension a building for vertical loading only and then just verify member capabilities under horizontal loadings. Individual structural members, have already code prescriptions which relate flexural and shear designs, but there are no explicit expressions relating vertical loading to horizontal loading for the building as a whole (Systems Approach). When we apply the expressions developed here, which contain dimensional and non-dimensional parameters, one finds that there must be certain relationships between the total vertical resistance and the total lateral force capabilities of the building. The two of them are interdependent. What comes out shows that the most important parameter for seismic design is the safety factor for vertical loading, which has to be increased when lateral loading increases. This fact is particularly important for large-panel concrete structures, masonry structures and cast-in-place shear walls, in that order. Framed structures depend on these parameters in a minor degree. The minimum sizes of the supporting members can be determined at the start of the project, the expressions giving the necessary conditions to be satisfied. If they are not, no amount of design will correct an initial bad decision. These expressions, were developed observing real life projects (about 100 of them), and statistical results derived from the Caracas 1967 earthquake. They have been applied with success in the classroom, giving the new engineers specific predimensioning tools.
Reference: "Predimensionamiento de Edificios Altos de Concreto Armado". Mario Paparoni M. Ediciones Sidetur, Caracas, Febrero de 1991.

INTRODUCTION

Structural Engineers customarily do not deal with non-dimensional parameters on their everyday calculations. The Author's experience with Structural Models taught him that by having a general view of the structure as a System, instead of an assemblage of members, leads to more direct and quicker ways of understanding their behavioural patterns. [Paparoni, 1991, Book].

The Caracas 1967 earthquake offered a very good opportunity to start and test this approach, when selected samples of surviving and damaged or fallen structures were studied in detail [Comisión Presidencial del Sismo, 1978]. (see pages 33 to 56). What came out was not surprising, the best correlated single index of performance was the mean safety factor of columns under vertical loading. Since then, efforts were devoted to develop other non-dimensional parameters to predict building performance and to allow good initial decisions in the projects. What follows is a selection of some of the parameters found

[1] *Universidad Metropolitana. Universidad Católica Andrés Bello. Caracas; Venezuela e-mails: mpapar@reacciun.ve*

BASIC CONCEPTS

The mean vertical safety factor of a building can be defined in the following way, taking into account all the supports (columns or walls)

$$\mathbf{F.S = \Sigma bhf'_c / W} \qquad (1)$$

Where W is the Seimic Weight of the Building (25% live load included), b and h the column sides (rectangular columns), f'c the mean concrte strength, reinforcement being ignored in this formula. The inverse of this Safety factor is the specific non-dimensional loading of a column (interaction diagrams) : **n=1/F.S.** If we look at the column as an individual member, we can relate several properties capable of being described by this parameter, together with its companion in Interaction Diagrams, the Specific Moment.

$$\mathbf{m = M/ bh^2 f'_c} \qquad (2)$$

These two indexes can describe and bound specific behavioural patterns, like rotational ductility, degree of cracking, type of failure, section shape dependence, balanced conditions, influence of reinforcement configuration, etc., from this idea it was decided to use these indexes, taken globally in a convenient statistical way, as global parameters of building behaviour, by simply taking as **W** the total weight of the building above the examined level, and **Σ bh** as the total column area (**a**) at that level, $\mathbf{f'_c}$ being the mean concrete strength. These expeessions can also be applied to concrete walls or to masonry.

As a part of the studies performed after the Caracas 1967 Earthquake on a sample of 127 selected buildings [Comisión Presidencial para el Estudio del Sismo Caracas, 1967] analyses were performed with several indexes, including (1) , the best correlation found to classify buildings according to their degree of damage, was found with the index **n,** when n>0.27 damages increased sharply. The 5 collapsed buildings had values of **n** between 0.30 and 0.45. ; m was not used in these studies, but later became a good practical index to evaluate at first instance building capabilities, when n>0.12, there is too much steel in the columns, one of the clues of insufficient dimensioning.
For framed buldings built before the 1967 Earthquake, the mean value of n was 0.25. When the Provisional Code was enforced, the ,mean value of n decreased to 0.20., as a result of the Drift limits imposed. [SABAL, Elvira, 1976].

From these ideas, and remembering the old galilean concept of the "weakness of the giants", it was very easy to relate two specific stresses which can be used to define the intensity of vertical loading produced by a building:

1) Mean soil pressure exerted by the building (Supposed as a prismatic body):

$$\mathbf{p=GH} \qquad (3)$$

Where: **p**: mean soil pressure over the ground floor area; **G**: Building unit weight (by volume); H: Building height.

2) Equivalent maximum uniform pressure bearable by the building supports (columns, walls)

$$\boldsymbol{\sigma} = \boldsymbol{\alpha}\mathbf{nf'_c} \qquad (4)$$

where: **σ:** equivalent maximum distributed pressure; **α**: support ratio = **a/A**, where **a**: total column (or supports) area; **A**: total floor area; and $\mathbf{nf'_c = f_c}$ (concrete working stress)

If we write an equilibrium equation relating (3) and (4) **(p=σ)** , we can deduce the following relationships:

$$\boldsymbol{\alpha} = \mathbf{GH/nf'_c} \qquad (5)$$

which gives the necessary ratio between column area and floor area to achieve a certain safety factor [1/n]
Rearranging (6), we get:

$$\mathbf{n = GH/}\boldsymbol{\alpha} \qquad (6)$$

which can be used to quantify the safety factor in an existing building. H and G can also be found in this way, giving maximum building height or unit weight of the building. (fixing the other parameters) Notice: The values of G for a Framed Concrete Buildings are of the order of 300 kgf/m^3 which goes up to 350 kgf/m^3 for Shear Wall Buildings and to 400 kgf/m^3 for Prefabricated Large Panel Structures [Sabal, 1991]. It is a better estimator than weight per unit area, because of its smaller variance and because it entails 3 dimensions and not 2. These values obviosly may change for cities different from Caracas.

So, we can say that if a column changes behaviour with **n** , so does a building. Low values of n for a building increase its resistance to earthquake loadings. (More Ductility, More reserves), as it is well known for columns.

RELATIOSHIP BETWEEN VERTICAL AND HORIZONTAL LOADING

Following the same reasoning, we can establish the necessary relationship between the vertical loading safety factor and the lateral loading safety factor, which practically determines the design of shear walls, concrete panels and masonry buildings, where only one fraction of the supports take shear and where shear strengths are no so high as in reinforced concrete columns.

If we define building weight as **W=anf'c,** being **W**= building weight, **a**: total column area at ground level, **n=f'c/f** ; f'c : concrete strength, then we can define the Acting Base Shear as

$$\mathbf{Qo = Cs*W = Cs*a*n*f'c} \qquad (7)$$

We can also express the upper limit of Base Shear which the building can resist as as:

$$\mathbf{Qmax = a*X*\kappa*f'c} \qquad (8)$$

Where **a**: total column (or wall) area; X: ratio between a_x and a_x+a_y (% of wall area in the X direction); $\kappa=v_u/f'c$,
Ratio between shear strength and compressive strength; f'c: concrete strength. There is a similar equation valid for the Y direction. Now, if we equate (7) and (8), and simplify, we get:

$$\mathbf{Cs*n = \kappa*X} \qquad (9)$$

We can deduce then:

$$\mathbf{Cs= \kappa X/n} \qquad (10)$$

Which can be worded as follows: The maximun Seimic Coeficient for Base Shear varies in proportion to the Shear Strength/Compressive Strength Ratio, in proportion to the percentage (X or Y) of supports (walls or columns) and directly in proportion to the Factor of Safety for Vertical Loading (1/n). This is a relationship entirely expressed by Non-Dimensional Parameters. It is, actually intuitive, but is forgotten in most cases and practically never mentioned in Seismic Codes.

We try to put confinement rules, minimum reinforcing rules, and so on, but we seem to forget that the seismic areas of the world intrinsecally demand higher Safety Factors for vertical loads, as one of the necessary requisites for structural viability. It means, simply that there must be an Achitecture for seismic areas, where lateral loading determines the dimensions of the supports, and not only vertical loading

It also shows that Framed Structures, where X = Y = 1, fare better in this condition than Concrete Shear Walls, Panels or Masonry, where, in the best case, if X=Y=0.50 and κ certainly has lower attainable values than framed structures. In the case of Prefabricated Panelled Buildings, the critical factor of attainable shear strengths in the joints puts a limit to their maximum heights. [Paparoni, 1984].

OTHER CONDITIONS

By applying the same procedures, other expressions can be derived as conditionals for design. Perhaps the simplest of them, valid for framed buildings is the mere fact that the sum of specific column loads has to be

limited, according to reinforcement configuration limits. We can say that $n_v + n_s$ and $m_v + m_s$ are bounded. nv being the specific load under vertical loading and ns the specific load for seimic loading.
In the case of $m_v + m_s$, m_v is usually very small, and m_s dominates. In the case of n_s, it is maximized in external columns or in end-of-the-chain for coupled walls. This contradicts the traditional approach of assigning smaller areas to peripheral columns, based on the traditional ideas of Tributary Area for Vertical Loading.

Some approximate expressions can be developed. For instance, ns for an outside column can be approximated by the expression

$$\mathbf{n_s = Q_o*(2/3)*H / B*a_f*f'_c} \qquad (11)$$

Where $\mathbf{Q_o}$: Base Shear; **H**: Building Height; **B**: Distance between opposite façades $\mathbf{a_f}$: Total façade columns area; $\mathbf{f'_c}$: Concrete Strength.

The maximum value for the sum $n_v + n_s$ can be estimated by multiplying $\phi = 0.7$ (column axial factor) by $\phi = 0.85$ (concrete equivalence coefficient) and by $\phi = 0.80$ (Axial Loading Upper Limit), 0ne gets then 0.476 Pmax/af'c, a value close to two times the mean value for Caracas Buildings constructed before 1967. Strangely enough, Codes do not explicitly limit this value, even accepting that column ductility decreases drastically for those loading levels.

It must be said that even if Pmax for a Reinforced Concrete Column increases with increasing longitudinal steel contents, ductility does not increase in the same proportion. As a matter of fact, the maximum acceptable value for nv+ns should be a function of the degree of confinement of the column. This parameter is normally not stated as a Code Limitation, except in a few isolated cases. (an old New Zealand Code).

Drift: It can be expressed in non-dimensional form by writing (for framed structures):
,

$$\mathbf{D = Factor^* (Q/C)} \qquad (12)$$

D: Drift ; Factor: depends on the Frame configuration; **Q** Base Shear; **C**: Shear Rigidity .

This non-dimensional parameter is now a standard criterion for seismic design. [Kameo, 1988], and many codes rely heavily on this limitation to control the dimensioning of building members (especially with framed structures).

Other useful non-Dimensional Factors can be be defined, for framed structures as the Momentality Factor and the Axiality Factor, ϕ mom = Total Moment absorbed in the column bases (ground floor) divided by the Overturning Moment. The axiality factor can be defined as 1-ϕ mom.

These parameters define the degree of coupling in framed structures. [Melandri,1991; Paparoni, 1981, 1984, 1990, 1991], making it possible to map the behavioural fields for framed structures, and to introduce sensitivity concepts into the design (sensitivity to earthquake-induced changes decrease with increased coupling). Coupling is basically a function of the ratio Shear Rigidity to Flexural Rigidity in Frames.

A more complete set of these parameters can be found in [Paparoni, 1991]., together with procedures to apply them in the Predimensioning of Tall Concrete Buildings. (Book). Sensitivity issues are discussed by [Paparoni, 1999]

REFERENCES

1) "Kameo, Aharón; Sanz, Carlos". Problemas de Deriva Sismica en Edificios. Estudio Paramétrico de las Variables Sistémicas que inciden sobre la Deriva de un Pórtico. Undergraduate Thesis . Universidad Metropolitana. Caracas. Sept. 1988.(Tutor: M.Paparoni).
2) "Melandri Majonica, Mónica". Verificación Matricial de Variables Configuracionales de Estructuras Aporticadas. Undergraduate Thesis. Universidad Metropolitana. Caracas.September 1991
3) "Ministerio de Obras Públicas. Comisión Presidencial para el Estudio del Sismo".
Segunda Fase del Estudio del Sismo Ocurrido en Caracas el 29 de Julio de 1967. Volúmenes A y B
FUNVISIS . Fundación Venezolana de Investigaciones Sismológicas. Caracas. Venezuela. 1978.
Vol. A 517 pages. Volumen B 519- 1281 pages.

4)” Paparoni M., Mario”. Prefabricados de Grandes Paneles. Limitaciones a los Coeficientes de Trabajo a Carga Vertical Impuestos por las Limitaciones resistentes a Cargas Horizontales en las Juntas. IV Congreso Venezolano de Ingenieria Sísmica. Barquisimeto. Venezuela. 1984.
5) “Paparoni M., Mario”. Evaluación Rápida de la Vulnerabilidad Sísmica de Estructuras de Edificios. Congreso UPADI 90. Washington D.C. 1990.
6) “Paparoni M., Mario”. *Dimensionamiento de Edificios Altos de Concreto Armado*. Book. Sidetur, Caracas. February 1991.
7) “Paparoni M., Mario”. “ Predimensionamiento de Estructuras Altas de Concreto Armado”. *Boletín IMME*, Año 21. Número 72-73. Enero-Diciembre 1981. pp. 102 to 164. Caracas. Universidad Central de Venezuela.
8) “Paparoni M. Mario”. What makes Framed Structures Well Suited to earthquake areas?. VI Congreso Venezolano de Sismología e Ingeniería Sísmica. Mérida, Venezuela. May 1999. Universidad de Los Andes. Mérida, Venezuela.
9) “Sabal Viana, Elvira C.”. Desarrollo de Criterios de Predimensionamiento para Edificios Altos. Undergraduate Thesis. Universidad Católica Andrés Bello. Caracas. 1976.