Figure 16 – Sprint World Headquarters, Kansas.

Figure 17 – 505 Fifth Avenue, NYC. (Photo Courtesy – Kohn Pederson Fox, NYC)

Figure 18 – Paramount Apartments.

Figure 19 – Proscenium Office Tower.(Courtesy: Case & Uzun, Atlanta, GA.)

Figure 20 – The Four Seasons, Miami Fla. (Courtesy: DeSimone Consulting Engineers.)

Figure 21 – Trump World Tower. (Courtesy: Trump Organization.)

Modeling Assumptions for Lateral Analysis

by J.F. Horvilleur, V.B. Patel, and K.A. Young

Synopsis: Reinforced concrete buildings must be proportioned to satisfy three limit states, serviceability, ultimate strength, and stability under sustained loads. This paper includes a detailed discussion of the recommended procedures and assumptions to be used in the design of reinforced concrete buildings for wind loads at these various limit states. Definition of the appropriate lateral load intensity, consideration of the structural parameters to be considered in the analysis, and discussion of suitable acceptance criteria is included. Differences in member properties at the limit states are prescribed based on variations in the degree of member cracking that can be expected at the load levels under consideration. The accurate prediction of the lateral stiffness of flat slab frames is also discussed. A summarization of the proper procedure and parameters to be used in the analysis of second order effects (P-Δ) is provided. Various other parameters affecting the analyses of buildings under sustained loads are addressed, including beam-column joint stiffness, foundation fixity, etc.

Keywords: lateral loads; limit states; member properties; reinforced concrete; second-order effects; serviceability; stability; stiffness; structural analysis

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INTRODUCTION

Reinforced concrete buildings must be proportioned to satisfy three limit states, serviceability, ultimate strength, and stability under sustained loads. An accurate analysis for each of these limit states requires the definition of the following in order to evaluate the structure:

1. Lateral load intensity
2. Frame stiffness
3. Acceptance criteria to be used

In this paper, all three issues are discussed. The authors offer a useful interpretation of the code requirements of lateral load analysis for reinforce concrete buildings, incorporating practical experience.

LATERAL LOAD INTENSITY

The lateral load intensity used in each analysis must be commensurate with the loads seen by the structure for each limit state. For the serviceability limit state, a wind load consistent with a service level condition is appropriate. It is within the judgment of the engineer to choose the lateral load to be used for this analysis, as this load is not code governed. The authors of this paper have successfully used 10-year winds to satisfy serviceability limit state on many buildings. Others have used a higher recurrence period wind to satisfy the serviceability limit state. Factors such as the type of building, types of occupants, the owner's expectations, and local wind climate can influence the selection of an appropriate recurrence period for wind. The approximate wind velocity for any recurrence period can be obtained from the commentary of ASCE 7-05 [1]. If a wind tunnel study is conducted based on the local climatic conditions, the wind load for the desired return period can be obtained by a more comprehensive evaluation of the local wind climate.

Although wind loads for the serviceability limit states are not code governed in the United States, all building codes define the wind loads for the strength limit state. When analyzing the structure for ultimate strength, the various building code (UBC, IBC2003, and ASCE 7-05) wind speeds correspond to a return period of roughly 50 years. To be exact, the ASCE 7-05 gives a value equal to the 720-year wind divided by the square root of 1.6, which is equal to the 50-year wind. Alternately, the results of a wind tunnel investigation may be used to define the appropriate wind loads that correspond to similar return period for the strength analysis.

When checking stability, the magnitude of lateral loads used is not significant and any set of lateral loads for X, Y, and torsional directions may be used. In actuality, this limit state does not have anything to do with wind. However, the stiffness of the lateral load resisting system of the building will need to be evaluated for the stability limit state. The evaluation of lateral stiffness of the frame involves estimating lateral deflection (drift) under a given set of lateral loads. The process of estimating lateral stiffness for verifying stability involves the analysis techniques which are analogous to drift evaluation are discussed herein for completeness.

FRAME STIFFNESS

Overall frame stiffness is a function of various parameters. Some of the most significant parameters that must be considered in the analysis are as follows;

- *Individual Member Properties Including the Effect of Cracking at the Appropriate Load Level*
- *Modulus of Elasticity*
- *Second Order Effects*
- *Various Analysis and Modeling Assumptions Made in the Lateral Analysis*

Each of the parameters is discussed in detail.

Individual Member Properties Including the Effect of Cracking

Member properties used in the assessment of each of these limit states must be representative of the degree of member cracking that can be expected at load levels consistent with the limit state under consideration. Accordingly, the lateral stiffness used in the analysis for each of the three limit states of serviceability, strength, and stability is unique. ACI 318 [2] Section 10.11 indicates cracking factors to be applied against the gross moment of inertia for the strength limit state. For the stability limit state, the same factors are used:

- Columns – 0.7
- T-Beams – 0.35 (T-beam as defined in ACI Section 8.10)

- Slabs – 0.25
- Shear walls – 0.7 (uncracked) or 0.35 (cracked)

For the serviceability, the following factors are prescribed:

- Columns – 1.0
- T-Beams – 0.50 (T-beam as defined in ACI Section 8.10)
- Slabs – 0.36
- Shear walls – 1.0 (uncracked) or 0.50 (cracked)

Note that the cracking factors for serviceability analysis reflect a lower level of cracking. According to ACI 318 commentary section R10.11.1, these factors are obtained by multiplying the cracking factors for strength by 1/0.7 (1.43). Cracking factors are clearly prescribed in ACI 318 for mild reinforced members, while engineering judgment is required to define cracking factors for post-tensioned beams and slabs.

It is the authors' opinion that for post-tensioned slabs and beams, cracked properties used for mild reinforced slabs and beams should be increased by a factor of approximately 30% to recognize the beneficial effect of axial prestress on flexural stiffness. The suggested 30% increase is based on engineering judgment as the magnitude of permissible increase is not discussed in the code. Explicit recommendations should be made and incorporated into future versions of the code.

The selection of cracking factors to be applied to the moment of inertia in shear walls requires a two-step process. First, the lateral analysis for ultimate strength should be conducted using a wall moment of inertia of $0.70I_g$. If the factored moments and shears obtained from this analysis indicate that the wall will crack in flexure, the analysis must be repeated using a moment of inertia of $0.35I_g$ for the levels where flexural cracking will occur. If the analysis indicates that the factored moments are not large enough to produce flexural cracking, the analysis with $0.70I_g$ will be adequate. Flexural cracking will occur when the flexural stress at the extreme fiber exceeds the modulus of rupture. The flexural stress is equal to P/A ± Mc/I. The modulus of rupture is equal to $7.50\sqrt{f'c}$ (psi). The ultimate load condition of 0.9D ± 1.60W will generally be the most critical condition for flexural cracking. For the purpose of estimating the extent of cracking in a shear wall, it is prudent to determine the flexural stress due to the design wind load level as cracking due to a strong wind will reduce the flexural stiffness for the remaining life of the structure.

Issues such as shear cracking and axial cracking need to be addressed. Also further clarification is also needed in the area of flat plates as to what is the effective width of the slab for the lateral analysis of frames.

Flat slab structures must be transformed into an equivalent frame model for lateral analysis unless a more detailed and time consuming finite element analysis is performed. Currently, there are two types of methods available to create an equivalent frame model from a flat slab structure: the effective width method and the transverse torsional member method. The effective width method is preferred for design due to simplicity and compatibility with analysis programs. When the effective slab width concept is used, columns are modeled in a conventional manner and modeling of torsional links is not required. Slab flexural stiffness is represented in the effective width method by the following equation:

$$E_c I_{eff} = \beta\alpha\ I_g\ E_c$$

where: β= stiffness reduction factor due to cracking (specified by ACI 318)
α = effective width factor that accounts for the actual 2-way transfer of moment
I_g = gross slab moment of inertia based on the tributary panel width

ACI Section 10.11.1 provides guidelines for the effect of cracking for various structural elements. However, no recommendation is provided for slab effective width. Current design office practice in some firms is to assume that α= 0.5. A study performed by the authors provides a more accurate estimate of α based on actual slab and column geometry.

The study performed by the authors involved calculating slab effective width factors α for various bay sizes, bay aspect ratios, column sizes, and slab thickness as determined by finite element analyses. Span to depth ratios were established for typical mild reinforced slabs and post-tensioned slabs. Effective width factors for interior, edge (about each axis), and corner columns were reported for each bay and slab geometry.

The effective widths reported in the study were the result of linear elastic finite element analyses performed in SAFE. Gross slab properties were used in the models, and the footprint of the column was assumed to be rigid. 5 ksi (34.5 MPa) concrete was used for all analyses. The slab flexural stiffness was determined by applying a moment to the centerline of the column and measuring the resulting slab rotation at the column. Results of the finite element analyses showed a wide variation in slab effective width depending on the panel and column geometry. Effective widths generally increase as the ratios dc/L1 and L2/L1 increase (see Figure 1 for notation). The effective widths were mostly independent of slab thickness. Due to various column dimensions and slab panel aspect ratios (L2/L2) found in practice, it is important to note that using α = 0.5 results in an effective width that may be inaccurate by **as much as 80%**.

Flat plate structures of equal spans were the only slab system modeled in the study. In addition, only square columns were studied. Further detailed research is required to determine the applicability of the study to systems with beams, drop panels, waffle slabs, and/or rectangular columns, and it is likely that similar studies performed for these systems will provide different results. Though the results may not be strictly applicable to other systems, they do provide a general indication of the large degree of variance in slab effective width that is possible based on manipulation of the floor framing layout. Alternatively, the slab can be modeled using finite elements in the lateral analysis model with proper cracking factors. With the advances in the computer analysis software as well as the speed of computers, this task is becoming easier than ever before.

Example results for flat slab systems of varying geometries are given in Table 1. The effective width factors given in the table should be combined with the cracking factors prescribed in ACI 318 to produce slab stiffnesses for lateral analysis. The effective width reduction factor α for post-tensioned slabs should be the same as that used for conventionally reinforced slabs.

An analysis of the finite element effective width results was performed to determine whether a trend exists in the data shown in Table 1. It was determined that when $b_{eff}/L1$ was plotted against the non-dimensional parameter $d_cL2/L1^2$ a reasonable curve could be fit through the data. Figure 2 shows the best fit curve for two conditions; the interior column and the edge column (where the edge is perpendicular to the lateral load), and the edge column (where the edge is parallel to the lateral load) and the corner column. Shown below the figure are the best fit equations for both cases.

The importance of accurately predicting the lateral stiffness of flat slab frames is emphasized in the commentary to ACI 318 Section 13.5.1.2. Essentially, this section states that a range of slab stiffnesses should be considered in design. Because typical flat slab buildings contain shear walls to provide the primary lateral load resisting system, underestimating the stiffness contribution of the slab system can lead to slab moments that may be too low, which could potentially result in a punching shear failure. Overestimating the slab system stiffness may inadvertently reduce both the lateral force delivered to the shear walls and the calculated drift.

Building systems that combine shear wall and slab systems should be analyzed twice for ultimate strength with a range of assumed slab stiffness. To determine shear wall forces, the effective widths reported in the study combined with the appropriate cracking factors will provide a good lower bound estimate of slab stiffness. To determine slab moments, particularly for punching shear checks, an upper bound estimate of slab stiffness should be used. There is no literature available to provide guidance on this subject and the upper bound stiffness assumed by different practicing engineers varies between 1 to 2 times the lower bound stiffness.

For interior slab panels I_g, should be based on the full panel width. For exterior panels and for frame action in a direction parallel to the edge of the building, I_g shall be based on one-half of the panel width. The effective width is required to take into account the fact that the stiffness of the entire panel is not mobilized under lateral loads.

ACI 318 only addresses flexural properties. Explicit recommendations are required for shear deformations, including the effects of shear cracking, and axial deformations in columns (including recommendations to calculate axial stiffness of columns in direct tension).

Modulus of Elasticity

Modulus of elasticity is calculated as prescribed in ACI-318 Section 8.5 for the serviceability and strength limit states, while the creep modulus of elasticity should be used when checking stability. The creep (or long-term) Modulus is obtained by dividing the elastic modulus by $1+\beta_d$. The reason for using the elastic modulus of elasticity for serviceability and strength limit state is that the lateral loads generated as a result of wind are of a transient nature.

Second Order Effects

The story P-Δ effect is caused by lateral deflections within the building frame due to the applied loading that result in additional internal forces and lateral deflections that are compounded with the forces and deflections found from an ordinary first order analysis. Computer programs for structural analysis use two different techniques to incorporate the P-Δ effect. The first method incorporates the P-Δ effect by modifying the structure stiffness matrix by the geometric stiffness. In this method, the analysis results will maintain the equilibrium. Therefore, the frame story shear will be equal to the applied story shear. In order to obtain the P-Δ effect, the analysis results such as lateral drifts, member forces, etc at any given location with and without P-Δ must be compared.

The second method used by many analysis software programs is non-iterative and is generally used for building type structures only. In this method, the additional overturning due to P-Δ is considered by additional story shear that is a function of the total weight above any story and the lateral displacement of that story. In this method, the resulting story shear in the lateral analysis is higher than the applied story shear. By simply comparing the frame story shear and applied story shear, the magnitude of the P-Δ effect can be obtained.

For a given building structure, the P-Δ effect at any story is a function of the weight of the structure above that level and the story stiffness. ACI 318 has clearly defined the member properties that can be used in the second order analysis as discussed above in order to properly estimate the story stiffness. Another factor that must be properly considered in the analysis is the weight of the building to be used in the P-Δ analysis. In the serviceability analysis, the weight to be used should be the sustained building weight, meaning the best estimate of the actual weight of the building. The sustained weight includes all fixed loads in the building, such as the weight of the floor slabs, beams, girders, columns, shear walls, cladding, topping slabs, masonry walls, mezzanines, etc. In addition, a realistic allowance must be included for sustained superimposed loads such as the actual weight of partitions, ceiling, mechanical equipment (including ductwork, piping, etc.), and live load. ASCE 7-05 Table C4-2 provides values of sustained live loads for various occupancies. A realistic estimate of the actual average weight of partitions, ceiling, and mechanical elements must be added to these values. Total average sustained superimposed loads range from 12 to 18 psf (0.574 to 0.862 kN/m^2) for occupancies such as office, residential, hotels, and schools. For the strength limit state, the building weight to be used in the analysis is equal to $1.2D + fL$ (where $f = 0.5$ for all occupancies in which design live load L is less than or equal to 100 psf (4.79 kN/m^2), with the exception of garages or areas occupied as places of public assembly where f shall be taken as 1.0). The value $1.2D + fL$ is the same weight as that used in the design of the columns, with the dead load D equaling the full design dead load and the live load L being the reduced live load. The load factors used in this combination are consistent with the fact that the P-Δ analysis is being conducted for lateral loads due to wind.

When considering stability, the building weight to be considered in the analysis is simply 1.2D + 1.6L. Since the stability analysis is not explicitly tied to the ability of the building to resist lateral wind loads, ultimate gravity loads without the lateral load due to wind must be used.

The acceptance criteria for the P-Δ effect under strength and stability limit states are discussed later.

Various Analysis and Modeling Assumptions Made in the Lateral Analysis

In addition to the various parameters affecting the analyses of buildings under sustained loads that we have previously discussed, other modeling assumptions such as beam column joint stiffness, foundation fixity, etc. also affect the behavior of the building when subjected to lateral loads. The effect of these assumptions and some further recommendations by the authors are discussed below.

Modeling Of Beam - Column Joint Stiffness [3] – Lateral displacements in tall reinforced concrete buildings consist of two different types, the "flexural type" which is due to elongation and shortening of the columns, and the "racking type" which is due to flexural and shear deformations of the beams and columns and distortion of the beam column joint. Seven different components combine to produce the cumulative lateral deformations in the beam-column subassemblage. These components are discussed in an accompanying paper by Horvilleur, et al [4] on the various components of drift. The lateral analysis must consider all sources of deformation including those which are due to stresses occurring in beam-column joints.

Based on the degree of assumed rigidity in the beam-column joint, significantly different results for lateral deflections may be obtained from computer analyses of concrete frames. A review of recent and past literature finds very little guidance for the practicing structural engineer in regard to how much rigidity should be assumed in the joint. At the extreme ends of the spectrum are the centerline/fully flexible (0% rigid) and fully (100%) rigid analyses. It is completely within the judgment of the structural engineer to analyze the frame with rigid joints, with partially rigid joints, or with flexible joints. Based on studies performed by the author, it is recommended that the beam-column joint be considered 50% rigid for lateral analysis. Detailed information regarding the authors' recommendation of the use of a 50% rigid joint can be found in the accompanying paper by Horvilleur. Deflections calculated from analyses assuming infinitely rigid joints will always be a fraction of actual deflections, and this assumption should never be used in practice. The assumption of completely flexible joints will always result in conservative results, giving deflections larger than the real values.