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Modeling Assumptions for Lateral Analysis

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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.

correct & suitable amount as compare to someone else.

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.

(Comparing b/w wings having similar features.

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. (clean & exact

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 (avoiding risk, careful) 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.

Modeling of Foundation Stiffness – A very limited amount of material has been found in the literature regarding the assumed rigidity of building foundations for lateral analysis, and the issue is often not considered in detail by practicing engineers. The assumption of full fixity at the base of the column will result in under-estimated drift values, while analyzing the column as fully pinned will give conservative results for drift. Some intermediate degree of fixity is likely appropriate, but there is no consensus in the industry as to the magnitude of fixity or the method of calculation of fixity for different foundation systems. It is important to note that the lateral story stiffness of the first story of any building subjected to wind will be significantly affected by whatever degree of fixity is assumed.

The degree of rotational stiffness provided by a square spread footing varies considerably depending on the footing size. Based on formulas published in the ***PCI Design Handbook/Sixth Edition [5]***, the total rotation of the column base is defined as:

$$\phi_b = \phi_f + \phi_{bp} + \phi_{ab}$$

where ϕ_f denotes the degree of rotation between the footing and the soil, ϕ_{bp} is the rotation due to the bending in the base plate, and ϕ_{ab} is the rotation due to elongation of the anchor bolts. For the cast in place reinforced concrete structures, ϕ_{bp} and ϕ_{ab} are zero and the rotation of the base is the same as the rotation between the footing and the soil. A flexibility coefficient, γ_f can be defined for the footing such that:

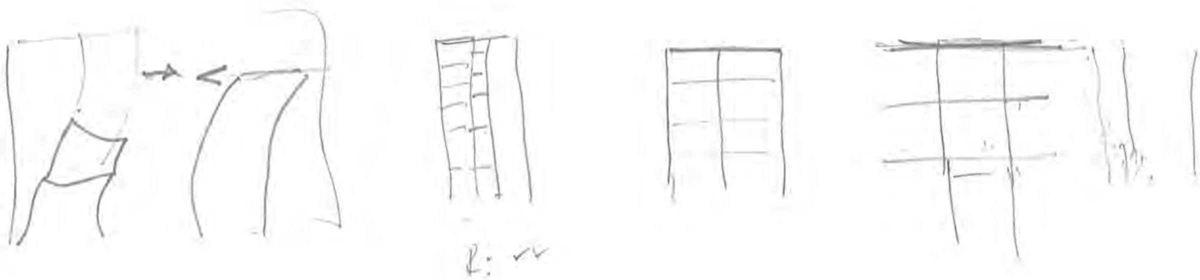
$$\gamma_f = \frac{1}{k_s I_f}$$

where k_s is the coefficient of subgrade reaction and I_f is the moment of inertia of the footing. Using the flexibility coefficient γ_f, the rotational stiffness K is expressed as:

$$K = 1/\gamma_f.$$

Rotational stiffnesses were tabulated and plotted for various footing sizes over a range of coefficients of subgrade reaction, k_s, as shown in Figure 3. Results were calculated for square footings varying in size from 8'-0 x 8'-0 (2.43 m x 2.43 m) to 20'-0 x 20'-0 (6.10 m x 6.10 m), with k_s values ranging from 100 pci to 240 pci (2.768×10^6 to 6.643×10^6 kg/m^3). Significant differences in rotational stiffness can be noted for the different footing sizes as the stiffness is proportional to the moment of inertia of the footing. As an example, a 20'-0 x 20'-0 (6.10 m x 6.10 m) footing has a rotational stiffness that is more than twice that of a 16'-0 x 16'-0 (4.88 m x 4.88 m) footing and sixteen times that of a 10'-0 x 10'-0 (3.05 m x 3.05 m) footing. The Geotechnical engineer should be consulted to determine the effect of the size of the foundations on the coefficient of subgrade reaction.

It should also be noted that the value of the coefficient of subgrade reaction is very important when assessing the rotational stiffness of the foundation. Soil subjected to



transient loading responds much differently than soil subjected only to sustained loads. The coefficient of subgrade reaction under sustained load should be smaller than that for the transient loads. This should be reflected when selecting the rotational spring stiffness of the foundation. If the rotation of foundation is predominantly from the lateral loads, the value of K based on transient loading may be appropriate for serviceability and strength limit states. However, if significant sway is expected under gravity loads or when the stability limit state is verified, a value of K based on the sustained load should be used. The recommendation of a geotechnical engineer should be obtained to assist in determining the appropriate coefficient of subgrade reaction.

A review of recent and past literature indicates that the issue of foundation stiffness, as it pertains to the lateral analysis of concrete buildings subjected to wind loads, has not been the subject of significant research efforts to date. A greater understanding of the effect of foundation rigidity is required such that practicing engineers will have more guidance in modeling foundation stiffness. Without more refined techniques, inaccuracies will continue to be inherent in lateral analyses due to errors in the assumed lateral stiffness of the lower levels of buildings and erroneous values for drift.

ACCEPTANCE CRITERIA

Each of the three limit states of serviceability, strength, and stability, require a different acceptability criteria to evaluate the performance of the building when subjected to lateral loads. For the serviceability limit state, an interstory drift limitation of H/400 has traditionally been used. Based on experience, this magnitude of drift is an acceptable amount of racking such that damage to non-structural elements will be minimized. This limit must be satisfied at all stories and at all plan locations within the story. Torsional effects must also be considered. Similar to the serviceability limit state, there is no code prescribed acceptability criteria for drift at strength load levels when evaluating the performance of a concrete building subjected to lateral loads.

The authors suggest a limit on the interstory stability index, $Q=P\Delta/Vh$ to be less than or equal to 0.25. This magnitude of Q indicates second order effects of 33%, which would be high enough to suggest that stiffening of the concrete frame should be considered. A code requirement has been placed on the interstory stability index calculated from stability analyses since ACI 318-95. A maximum value of $Q=0.6$ has been prescribed in the code and this value should never be exceeded. This value of Q indicates a P-Δ effect of 150% and is equivalent to a factor of safety of 1.67 against instability under sustained loads. This limit state also must be checked at all floor levels.

Traditionally, drift is calculated by finding the relative lateral displacement of two consecutive levels and dividing it by the story height. This lateral drift is then compared with the acceptance criteria. However, in some cases, the drift calculated using this method may not be of much significance. Errors using pure drift indices will be high when the axial deformations are significant, as they are for tall buildings.

If the goal in defining a drift limit is limited to only the control of damage to collateral building elements, such as cladding and partitions, and is separated from the problem of building motion, then frame racking or shear distortion (strain) is the logical parameter to evaluate.

Mathematically, if the local x, y displacements are known at each corner of an element or panel, then the overall average shear distortion for rectangular panel ABCD as shown in Figure 4 may be termed the Drift Measurement Index (DMI, as defined by Griffis [6]) and defined as follows:

Drift Measurement Index (DMI) = average shear distortion

$$DMI = 0.5 \times [(X_A-X_C)/H + (X_B-X_D)/H + (Y_D-Y_C)/L + (Y_B-Y_A)/L]$$

$$DMI = 0.5 \times (D1 + D2 + D3 + D4)$$

where,

X_i = vertical displacement of point i
Y_i = lateral displacement of point i
$D1 = (X_A-X_C)/H$, horizontal component of racking drift
$D2 = (X_B-X_D)/H$, horizontal component of racking drift
$D3 = (Y_D-Y_C)/L$, vertical component of racking drift
$D4 = (Y_B-Y_A)/L$, vertical component of racking drift

It is to be noted that terms D1 and D2 are the horizontal components of the shear distortion or frame racking and are the familiar terms commonly referred to as Interstory Drift. The terms D3 and D4 are the vertical components of the shear distortion or frame racking caused by axial deformation of adjacent columns.

If it can be accepted that the DMI is the true measure of potential damage, then it becomes readily apparent that the evaluation of interstory drift alone can be misleading in obtaining a true picture of potential damage. Interstory drift alone does not account for the vertical component of frame racking in the rectangular panel that also contributes to the potential damage, nor does it exclude rigid body rotation of the rectangular panel which by itself does not contribute to damage. It can be shown that evaluation of the commonly used Interstory Drift can significantly underestimate the damage potential in a combined shear wall/frame type building where the vertical component of frame racking can be important; and significantly overestimate the damage potential in a shear wall building where large rigid body rotation of a story can occur due to axial shortening of columns.

It is logical to identify the rectangular panel ABCD in Figure 4 as the zone in which the damage potential is to be evaluated and define it the Drift Measurement Zone (DMZ). From a practical standpoint, these zones will typically represent column bays within a building and would be incorporated as part of the building frame analysis.

Once the determination of the shear distortion or Drift Measurement Index (DMI) is made for different column bays or Drift Measurement Zones (DMZ's), it must be compared to a damage threshold value for the element being protected. These damage threshold limits can be defined as the shear distortion or racking that produces the maximum amount of cracking or distress that can be accepted, on the average, once every ten years. Note that depending on the type of building and owners' expectations, it may be necessary to compare DMI to the damage threshold that corresponds to a longer return period. It is logical to define these damage threshold shear distortions as the Drift Damage Index (DDI). From the standpoint of serviceability limit states it is necessary to observe the following inequality:

Drift Measurement Index ≤ Drift Damage Index

$$DMI \leq DDI$$

A significant body of information is available from racking tests for different building materials that may be utilized to define DDI's, as discussed by Griffis.

Figure 5 shows the floor plan of a 10-story building studied by the authors. The structure of the building is an 8" (203 mm) post-tensioned slab with 24"x24" (610 mm x 610 mm) columns and four 12" (305 mm) thick x 20'-0 (6.1 m) long shear walls to resist lateral loads.

Floor-to-floor heights were set at 10'-0 (3 m). DMI values were calculated for a serviceability load case using the 10-year wind and Exposure B as shown in Figure 6. The ratio of DMI/story drift is shown in Figure 7.

Note the large variation in DMI for Panels 1-3, with Panel 2 (Gridline A, between 2-3) having the largest DMI and Panel 3 (Gridline A, between 1-2) having the smallest DMI. This location coincides with the placement of the shear walls in the building, where DMI and drift are essentially equal.

For convenience, Table 2 provides a summary combining the code-mandated analysis provisions and suggested analysis parameters that are provided by the authors in the sections above.

SUMMARY

In order to perform an accurate and complete lateral analysis of a reinforced concrete building, many different parameters must be considered. The key factors to be considered when modeling the behavior of a reinforced concrete frame subjected to lateral loads can be summarized as follows:

1. The appropriate limit states must be considered; serviceability, strength, and stability. Lateral load levels consistent with each limit state are to be used in each analysis.

2. The frame stiffness used in the analysis must consider the degree of cracking of different types concrete members, the member properties, second order effects, beam-column joint stiffness, foundation stiffness, and other factors.
3. The acceptance criteria used to evaluate the performance of the building at each limit state should be based on code requirements or, in the absence of prescribed code limits, engineering judgment. Damage to collateral building elements, such as cladding and partitions, should be considered as appropriate based on the desired performance of the building.

ACKNOWLEDGEMENTS

Prior to his death in September, 2002, the conceptual development of this paper was lead by Mr. Javier F. Horvilleur. Mr. Horvilleur passed away before this paper was completed and the authors wish to dedicate this paper to Mr. Javier F. Horvilleur.

NOTATION

A	:	Area
A_{ac}	:	Gross axial area of column.
A_{acr}	:	Effective (cracked) axial area of column.
A_v	:	Area of shear reinforcement
A_{vcr}	:	Effective (cracked) shear area of column, beam, or slab.
A_{vg}	:	Gross shear area of column, beam, slab, or shear wall.
b	:	width of column
b_w	:	width of beam
c	:	Distance from compression face to neutral axis
D	:	Design dead load.
d_c	:	depth of column
d_b	:	depth of beam
E_c	:	Modulus of elasticity of concrete.
Fx	:	Lateral load in x direction applied at the center of mass at the roof level.
Fy	:	Lateral load in y direction applied at the center of mass at the roof level.

f'c	:	Concrete compressive strength
h	:	Story height.
I_{eff}	:	Effective moment of inertia
I_f	:	Moment of inertia of footing
I_g	:	Gross moment of inertia of column, beam, slab, or shear wall.
K	:	Rotational Stiffness
k_s	:	Coefficient of subgrade reaction
L	:	Design live load including live load reduction allowed by the general building code.
M	:	Bending moment
n	:	Modular Ratio; $n = E_s/E_c$
P	:	Axial Load
P_{cr}	:	Axial Load at which cracking occurs
Q	:	Story stability index equal to $P\Delta/Vh$.
s	:	Spacing of shear reinforcement
$T\theta$	:	Torsional load applied at the center of mass at the roof level.
V_c	:	Nominal shear strength provided by concrete
V_u	:	Factored shear force
W	:	Design wind load
α	:	Effective width reduction factor.
β	:	Stiffness reduction factor due to cracking
β_d	:	Ratio of maximum story factored sustained axial load to total story factored axial load.
$\varnothing_b$	:	Rotation of column base
$\varnothing_f$	:	Rotation between footing and soil

$\o_{bp}$: Rotation due to elongation of anchor bolts

$\o_{ab}$: Rotation due to elongation of anchor bolts

Δ : Inter-story deflection.

γ_f : Flexibility coefficient

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Table 1A – Effective Width Factors for Flat Slab Systems
Typical Mild Reinforced Slab Span to Depth Ratios

Typical Mild Reinforced Slab Span to Depth Ratios				Interior	Edge Slab: Load Perpendicular to Edge	Edge Slab: Load Parallel to Edge	Corner
Slab T	L1	L2	Col. Size	α	α	α	α
in	ft	ft	ft				
8	24	24	1	0.39	0.41	0.40	0.42
8	20	24	1	0.46	0.48	0.47	0.50
8	16	24	1	0.55	0.57	0.56	0.59
8	12	24	1	0.67	0.69	0.69	0.71
8	24	20	1	0.34	0.36	0.36	0.38
8	24	16	1	0.30	0.32	0.32	0.33
8	24	12	1	0.24	0.26	0.27	0.28
8	24	24	2	0.48	0.51	0.51	0.54
8	20	24	2	0.56	0.59	0.59	0.62
8	16	24	2	0.67	0.69	0.70	0.72
8	12	24	2	0.79	0.80	0.82	0.83
8	24	20	2	0.43	0.46	0.47	0.49
8	24	16	2	0.37	0.40	0.42	0.43
8	24	12	2	0.31	0.33	0.36	0.36
8	24	24	3	0.55	0.58	0.59	0.61
8	20	24	3	0.63	0.66	0.68	0.69
8	16	24	3	0.74	0.75	0.78	0.79
8	12	24	3	0.85	0.85	0.88	0.88
8	24	20	3	0.49	0.52	0.54	0.55
8	24	16	3	0.42	0.46	0.49	0.49
8	24	12	3	0.35	0.38	0.42	0.41
8	24	24	4	0.60	0.63	0.65	0.66
8	20	24	4	0.69	0.71	0.74	0.74
8	16	24	4	0.79	0.80	0.83	0.83
8	12	24	4	0.89	0.88	0.92	0.91
8	24	20	4	0.54	0.57	0.60	0.60
8	24	16	4	0.47	0.49	0.54	0.53
8	24	12	4	0.38	0.40	0.47	0.45

0.25x

x

Table 1A (Cont'd) – Effective Width Factors for Flat Slab Systems
Typical Mild Reinforced Slab Span to Depth Ratios

10	30	30	1	0.37	0.38	0.37	0.39
10	26	30	1	0.42	0.43	0.42	0.44
10	22	30	1	0.48	0.50	0.48	0.51
10	18	30	1	0.57	0.58	0.57	0.60
10	30	26	1	0.33	0.35	0.34	0.36
10	30	22	1	0.30	0.31	0.31	0.33
10	30	18	1	0.26	0.27	0.27	0.29
10	30	30	2	0.45	0.48	0.47	0.50
10	26	30	2	0.51	0.54	0.53	0.56
10	22	30	2	0.59	0.61	0.61	0.64
10	18	30	2	0.68	0.70	0.70	0.72
10	30	26	2	0.41	0.44	0.44	0.46
10	30	22	2	0.37	0.40	0.40	0.42
10	30	18	2	0.32	0.35	0.36	0.37
10	30	30	3	0.51	0.54	0.55	0.57
10	26	30	3	0.58	0.61	0.61	0.63
10	22	30	3	0.66	0.68	0.69	0.71
10	18	30	3	0.75	0.76	0.78	0.79
10	30	26	3	0.47	0.50	0.51	0.53
10	30	22	3	0.42	0.45	0.47	0.48
10	30	18	3	0.37	0.40	0.42	0.43
10	30	30	4	0.55	0.59	0.60	0.62
10	26	30	4	0.62	0.65	0.67	0.68
10	22	30	4	0.71	0.73	0.75	0.78
10	18	30	4	0.80	0.80	0.83	0.83
10	30	26	4	0.51	0.54	0.55	0.57
10	30	22	4	0.46	0.49	0.52	0.52
10	30	18	4	0.40	0.43	0.47	0.47

Note: 1 ft = 0.305 m
1 in = 25.4 mm

Table 1B – Effective Width Factors for Flat Slab Systems Typical Post-Tensioned Slab Span to Depth Ratios

Typical Post-Tensioned Slab Span to Depth Ratios				Interior	Edge Slab: Load Perpendicular to Edge	Edge Slab: Load Parallel to Edge	Corner
Slab T in	L1 ft	L2 ft	Col. Size ft	α	α	α	α
8	30	30	1	0.37	0.38	0.37	0.39
8	26	30	1	0.42	0.43	0.42	0.45
8	22	30	1	0.48	0.50	0.49	0.51
8	18	30	1	0.57	0.58	0.57	0.60
8	30	26	1	0.33	0.35	0.34	0.36
8	30	22	1	0.30	0.31	0.31	0.33
8	30	18	1	0.26	0.27	0.27	0.29
8	30	30	2	0.45	0.48	0.47	0.50
8	26	30	2	0.51	0.54	0.53	0.56
8	22	30	2	0.59	0.61	0.61	0.64
8	18	30	2	0.68	0.70	0.70	0.72
8	30	26	2	0.41	0.44	0.44	0.46
8	30	22	2	0.37	0.40	0.40	0.42
8	30	18	2	0.32	0.35	0.36	0.38
8	30	30	3	0.51	0.54	0.55	0.57
8	26	30	3	0.58	0.61	0.61	0.63
8	22	30	3	0.66	0.68	0.69	0.71
8	18	30	3	0.75	0.77	0.78	0.79
8	30	26	3	0.47	0.50	0.51	0.53
8	30	22	3	0.42	0.45	0.47	0.48
8	30	18	3	0.37	0.40	0.42	0.43
8	30	30	4	0.55	0.59	0.60	0.62
8	26	30	4	0.63	0.65	0.67	0.68
8	22	30	4	0.71	0.73	0.75	0.76
8	18	30	4	0.80	0.81	0.83	0.83
8	30	26	4	0.51	0.54	0.65	0.57
8	30	22	4	0.46	0.49	0.52	0.52
8	30	18	4	0.40	0.43	0.47	0.47

Table 1B (Cont'd) – Effective Width Factors for Flat Slab Systems Typical Post-Tensioned Slab Span to Depth Ratios

10	37.5	37.5	1	0.34	0.35	0.34	0.36
10	32.5	37.5	1	0.39	0.40	0.39	0.41
10	27.5	37.5	1	0.45	0.46	0.45	0.46
10	22.5	37.5	1	0.53	0.54	0.53	0.56
10	37.5	32.5	1	0.31	0.32	0.31	0.33
10	37.5	27.5	1	0.28	0.29	0.28	0.30
10	37.5	22.5	1	0.24	0.25	0.25	0.27
10	37.5	37.5	2	0.42	0.44	0.43	0.46
10	32.5	37.5	2	0.48	0.50	0.49	0.52
10	27.5	37.5	2	0.55	0.57	0.57	0.59
10	22.5	37.5	2	0.64	0.66	0.65	0.68
10	37.5	32.5	2	0.38	0.41	0.40	0.43
10	37.5	27.5	2	0.34	0.37	0.37	0.39
10	37.5	22.5	2	0.30	0.32	0.33	0.35
10	37.5	37.5	3	0.47	0.50	0.50	0.53
10	32.5	37.5	3	0.54	0.57	0.57	0.59
10	27.5	37.5	3	0.61	0.64	0.64	0.67
10	22.5	37.5	3	0.71	0.73	0.73	0.75
10	37.5	32.5	3	0.43	0.47	0.47	0.49
10	37.5	27.5	3	0.39	0.42	0.43	0.45
10	37.5	22.5	3	0.34	0.37	0.39	0.40
10	37.5	37.5	4	0.52	0.55	0.56	0.58
10	32.5	37.5	4	0.58	0.62	0.62	0.64
10	27.5	37.5	4	0.66	0.69	0.70	0.72
10	22.5	37.5	4	0.76	0.77	0.79	0.80
10	37.5	32.5	4	0.47	0.51	0.52	0.54
10	37.5	27.5	4	0.43	0.46	0.48	0.49
10	37.5	22.5	4	0.37	0.40	0.43	0.44

Note: 1 ft = 0.305 m
1 in = 25.4 mm

Table 2 – Summary of Analysis Parameters

Issue	Analysis I Serviceability Limit State	Analysis II Ultimate Strength Limit State	Analysis III Stability Under Sustained Loads Limit State
Lateral Load Intensity	(1) 10 yr wind	50 yr wind	(2) Fx = 1.0 kips Fy = 1.0 kips Tθ = 1.0 k-in
Member Properties			
Columns flexure	1.0 I_g	0.70 I_g	0.70 I_g
Columns Axial	(3) A_{gc}	(3) A_{acr}	(3) A_{acr}
Mild Reinforced Beams	(4) 0.50 I_g	(4) 0.35 I_g	(4) 0.35 I_g
Post-Tensioned Beams	(5) 0.67 I_g	(5) 0.45 I_g	(5) 0.45 I_g
Mild Reinforced Flat Slabs	(6) 0.36 α I_g	(6) 0.25 α I_g	(6) 0.25 α I_g
Post-Tensioned Flat Slabs	(7) 0.48 α I_g	(7) 0.33 α I_g	(7) 0.33 α I_g
Shear Walls Cracked Uncracked	(8) 0.50 I_g 1.00 I_g	(8) 0.35 I_g 0.70 I_g	(8) 0.35 I_g 0.70 I_g
Shear Deformations	(9) A_{vg}	(9) A_{vg}	(9) A_{vg}
Modulus of Elasticity	(10) $E = E_c$	(10) $E = E_c$	(11) $E = \dfrac{E_c}{1 + \beta_d}$
Building weight to be used for P-Δ analysis purposes	(12) Sustained Building Weight	(13) 1.2D+fL	(13) 1.2D + 1.6L
Beam-Column Joint Stiffness	(14) 50% Rigid	(14) 50% Rigid	(14) 50% Rigid
Acceptability Criterion	(15) Inter-story Drift $\Delta/h \le 0.0025$	(16) Inter-story Stability Index $Q \le 0.25$	(17) Inter-story Stability Index $Q \le 0.60$

Commentary to Table 2

Note 1. The 10 year wind speed may be obtained from ASCE 7-05 Commentary Table C6-7. Other recurrence intervals may be appropriate depending on the characteristics of the project.

Note 2. Any lateral load may be used in order to assess the Limit State of Stability under Sustained Loads. A single load of 1.0 kips (or 1 kN) in the X and Y directions and a torque of 1.0 kip-in. (or 1 kN-m) applied at the roof level and at the floor center of mass is recommended. Application of a unit torque is required in order to assess torsional stability.

Note 3. In the calculations of the gross axial area of the columns the contribution of the reinforcing steel is normally ignored. If the reinforcing steel is to be taken into account, the transformed axial area must be used.

In cases in which the building overturning moment results in net tension in some of the columns, the effect of axial cracking must be considered in the determination of the column axial stiffness. The equation for the cracked axial area A_{acr} was obtained from ACI Committee 224 [7] where all the variables are defined.

Column Axial Area		
Analysis I Serviceability Limit State	**Analysis II Ultimate Strength Limit State**	**Analysis III Stability Under Sustained Loads Limit State**
Gross Axial Area – Column $A_{ac} = b\ d_c$	Cracked Axial Area- Column $A_{ACR} = A_{AC}\left(\frac{P_{CR}}{P_U}\right)^3 + A_{CR}\left[1-\left(\frac{P_{CR}}{P_U}\right)^3\right]$	Cracked Axial Area- Column $A_{ACR} = A_{AC}\left(\frac{P_{CR}}{P_U}\right)^3 + A_{CR}\left[1-\left(\frac{P_{CR}}{P_U}\right)^3\right]$

Note 4. Beam gross moment of inertia I_g shall be based on the tee-beam section as defined by ACI 318 [2] Section 8.10.

Note 5. Cracked properties used for mild reinforced beams have been increased by a factor of approximately 30% to account for the beneficial effect of axial prestress on flexural stiffness. This increase is based on engineering judgment.

Note 6. 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. Unless a more detailed analysis is conducted, such as that performed by the authors, the effective width reduction factor α may be taken as 0.50. The effective width is required to take into account the fact that the stiffness of the entire panel is not mobilized under lateral loads. When the effective slab width concept is used, columns are modeled in a conventional manner. Modeling of torsional links is not required.

Note 7. Cracked properties used for mild reinforced slabs have been increased by a factor of approximately 30% to recognize the beneficial effect of axial prestress on flexural stiffness. The effective width reduction factor α should be the same as that used for conventionally reinforced slabs.

Note 8. The selection of cracking factors to be applied to the moment of inertia of shear walls requires a two step process. First, the ultimate strength lateral analysis (Analysis II) should be conducted using a wall moment of inertia of 0.70 I_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 0.35 I_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 0.70 I_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 $7.50\sqrt{f'c}$. The ultimate load condition of 0.90D ± 1.60W will generally be the most critical condition as far as flexural cracking is concerned. Analysis III should use the same cracking factors as those of Analysis II. Analysis I should use the cracking factors of Analysis II multiplied by 1.43.

Note 9. For structures with relatively large spans, shear deformations are usually responsible for only a small fraction of lateral drift of concrete buildings. This is particularly true for frames with 30 to 40 foot (9.1 m to 12.2 m)bays. In these cases, using gross shear areas A_{vg} in analysis will yield sufficiently accurate results. The effect of shear cracking on shear stiffness should be taken into account on tubular buildings which feature beams with small span to depth ratios. When shear deformations are important, the effect of shear cracking for serviceability, strength and stability can be considered as shown below.

The gross shear area is generally taken equal to 5/6 of the area of the beam stem. In the equations for A_{vcg} and A_{vbg}, b and b_w are the width of the columns and beams respectively with d_c and d_b being the depth of the same. If shear cracking is expected at load levels consistent with the serviceability limit state, the cracked shear areas should be used in the analysis.

Tests conducted by Dilger and Abele [8] indicate that diagonal cracking of the faces of concrete members result in a significant reduction of shear stiffness. These diagonal cracks form when the shear in the member is larger than the shear that can be sustained by the concrete alone. When the member shear V_u is smaller than V_c, diagonal cracking does not occur and the gross shear area may be used in analysis. Using the truss analogy, Dilger and Abele have derived equations to calculate the cracked shear area. The deformations predicted with the proposed equations compare very well with deformations measured in laboratory tests. In the case of beams, the cracked shear area A_{Vbcr} is a function of the dimensions of the beam b_W and d_b, the modular ratio n which is equal to E_s/E_c, the area of the stirrups A_V and their spacing s, of V_c which is equal to the shear capacity of the concrete section alone and of V_u which is the ultimate beam shear. In the case of the columns, the equation for A_{Vccr} is the same. In the determination of V_c for the columns, the beneficial effect of the axial compression must be taken into account. Using a similar expression the effective shear area of the beam-column joint can also be estimated. Note that V_u can be taken as maximum, since $V = 2M/L$ is function of only M (not M and P) excluding gravity loads.

Column Shear Area		
Analysis I Serviceability Limit State	**Analysis II Ultimate Strength Limit State**	**Analysis III Stability Under Sustained Loads Limit State**
Gross Shear Area – Column $A_{vcg} = 0.83\ b\ d_c$	Cracked Shear Area- Column $A_{VCCR} = \dfrac{2.14\ b\ d_c n}{\left(\dfrac{Vu - Vc}{Vu}\right)\left(\dfrac{b\ s}{Av}\right) + 4n}$	Cracked Shear Area- Column $A_{VCCR} = \dfrac{2.14\ b\ d_c n}{\left(\dfrac{Vu - Vc}{Vu}\right)\left(\dfrac{b\ s}{Av}\right) + 4n}$

Beam Shear Area		
Analysis I Serviceability Limit State	**Analysis II Ultimate Strength Limit State**	**Analysis III Stability Under Sustained Loads Limit State**
Gross Shear Area – Beam $A_{vbg} = 0.83\ b_w\ d_b$	Cracked Shear Area- Beam $A_{VBCR} = \dfrac{2.14\ b_w\ d_b n}{\left(\dfrac{Vu - Vc}{Vu}\right)\left(\dfrac{bw\ s}{Av}\right) + 4n}$	Cracked Shear Area- Beam $A_{VBCR} = \dfrac{2.14\ b_w\ d_b n}{\left(\dfrac{Vu - Vc}{Vu}\right)\left(\dfrac{bw\ s}{Av}\right) + 4n}$

Note 10. The elastic modulus of elasticity should be computed using ACI 318-05 Section 8.5.

Note 11. The long-term modulus of elasticity must be used in the assessment of stability under sustained lateral loads. This may be done by dividing the elastic modulus by $(1 + \beta_d)$, where β_d is the ratio of the maximum story factored sustained axial load to the total story factored axial load.

Note 12. The sustained building weight shall be computed as the sum of:

a. The fixed loads which consist of the weight of floor slabs, beams, girders, columns, shear walls, cladding, topping slabs, masonry walls, mezzanines, etc., and

b. A realistic allowance for sustained superimposed loads which include the actual weight of partitions, ceiling, mechanical, 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 needs to 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.

Note 13. The dead load D is the full design dead load. The live load L is the reduced live load. These loads are the same loads used to design columns. The load factors on Analysis II are consistent with the fact that the P-Δ analysis is being conducted with the wind lateral loads. For stability analysis the load factors are consistent with the fact that the P-Δ analysis is being conducted without the wind lateral loads. The value of f shall be equal to 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.

Note 14. Finite element analysis of the flexibility of beam column joints indicate that the best lateral frame stiffness correlation is obtained when the joints are assumed to be only 50% rigid. This means that the size of the rigid zone is assumed to be only one half of the actual size of the beams and columns framing to the joint. Additional background on this subject is provided in an associated paper by Horvilleur, Patel, and Young [4] on drift components.

Note 15. The maximum allowable inter-story drift of 0.0025 shall be satisfied at all stories and at all plan locations within the story. Torsional effects must be considered. If axial deformations are significant, the DMI should be used.

Note 16. This limit is not a code requirement. However, a story stability index of 0.25 will result in second order effects of 33%, which is relatively high and may indicate that the frame should be stiffened.

Note 17. This is a code requirement and should never be exceeded. A story stability index of 0.60 means that the story has a factor of safety of 1.67 against instability under sustained loads.

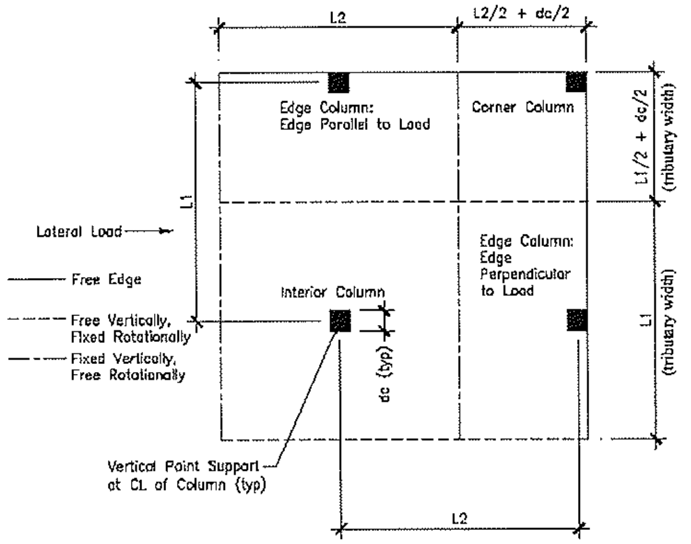

Figure 1 – Notation for flat slab effective width study

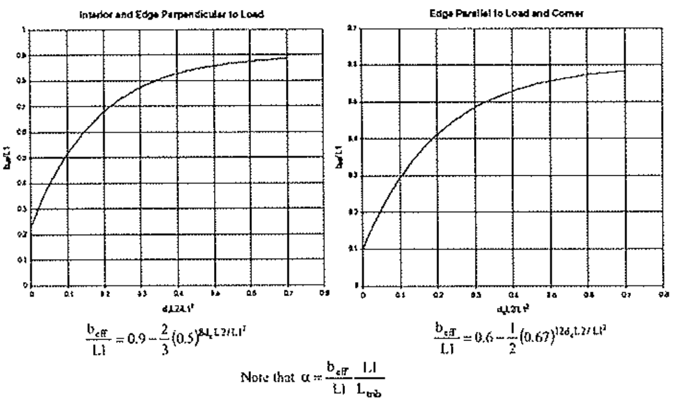

Figure 2 – Best-fit curves based on effective width study

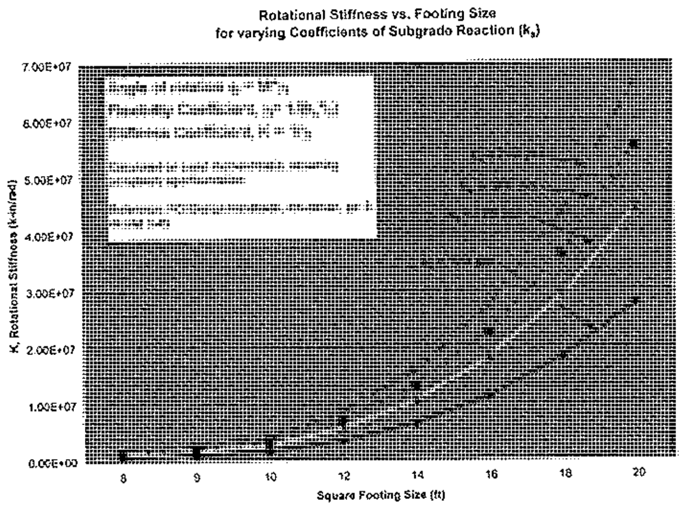

Figure 3 – Rotational stiffness as a function of square footing size
Note: 1 ft = 0.305 m, 1 pci = 27,679 kg/m3, 1 k-in./rad = 11,521 kg-m/rad

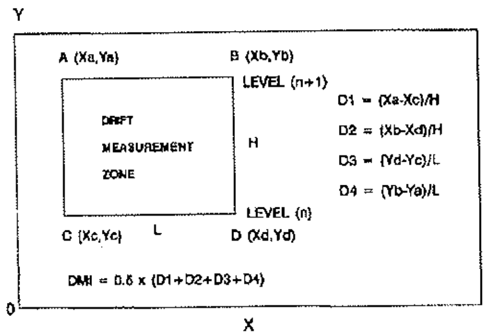

Figure 4 – Drift measurement index (DMI)

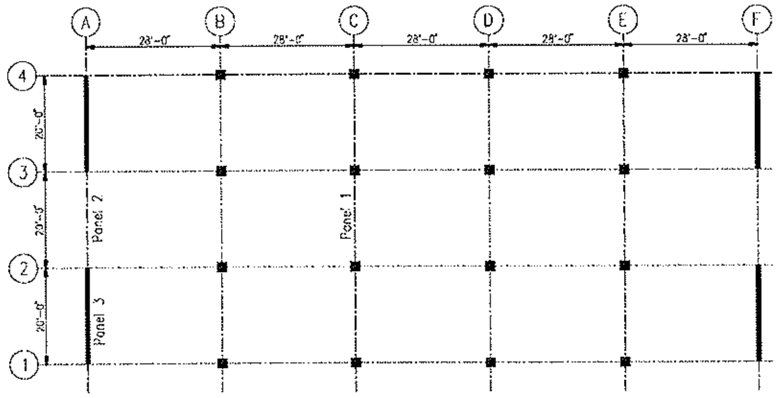

Figure 5 – Plan of building used in drift measurement index (DMI) study case
Note: 1 ft = 0.305 m

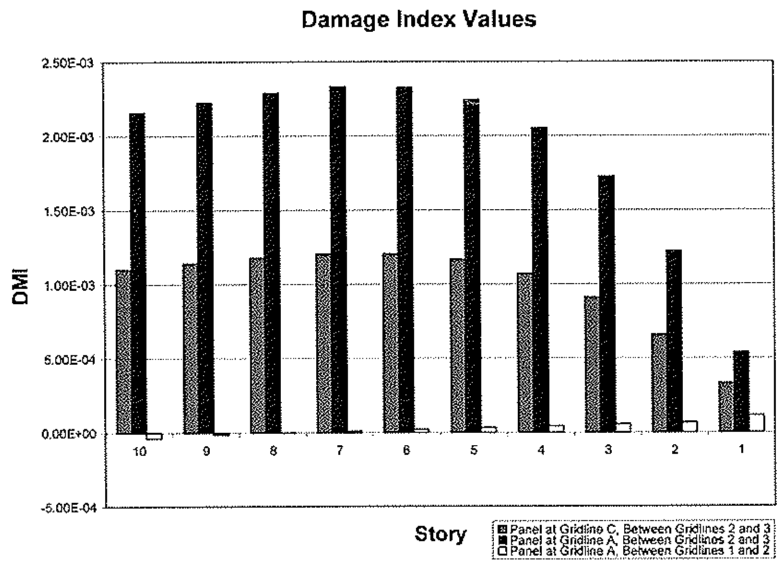

Figure 6 – Drift measurement index (DMI) values for study case building

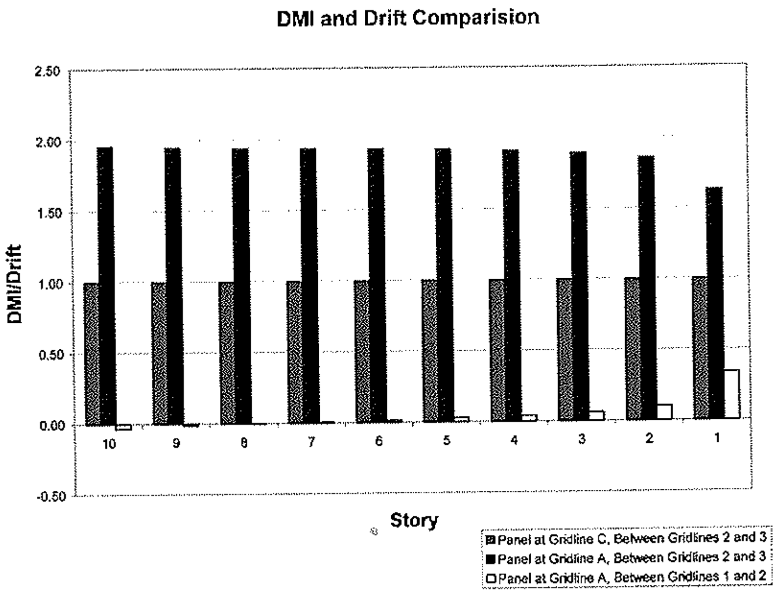

Figure 7 – Ratio of DMI/drift for study case building