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Verification of Proposed Design Methodologies For

Effective Width of Slabs In Slab-Column Frames

By: Jacob S. Grossman
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The design methodologies evaluated in this paper are as follows: the methodology proposed by the researchers of the experimental data [1]; an earlier design methodology [2] (which has been used since the late 1970's in the design of many structures by this author); and a permutation of this earlier design methodology.

A "rational" methodology to estimate the contribution of slab-column frames to the stiffness of 3-D structures at various lateral load levels of interest (serviceability, strength design, and limit states), along with the results of initial efforts to verify this methodology via measurements of high-rise flat slab structures, is presented.

Keywords: effective width; slab-column frames; flat plate; structure; serviceability; strength design; stiffness; drift limits; ductility; redundancy; dynamic properties; damping; wind; seismic; design methodologies.

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The designer must know the dynamic properties of the structure (its stiffness, mass and damping) in order to evaluate the lateral loads the structure will absorb during seismic or wind storm events, as well as to determine the structure's ability to successfully dissipate such action. In concrete structures, two of the three properties mentioned above (stiffness and damping) are variables dependent on time (history of events) and the load-level the structure is absorbing or has absorbed. The design process is iterative -- requiring a review of the structure's various building blocks for stiffness degradation at the load levels of concern (serviceability, strength design and limit states).

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Among the more common building blocks (columns, beams, shear-walls) flat slabs and plates are used extensively, most notably on the East Coast, in the design of high-rise apartment buildings. In these buildings, architectural constraints did not allow for the incorporation of many beams and shear walls in the design. Engineers, responding to the demand for this economical type of structure, have attempted to estimate the stiffness of the two-way slab and its contribution toward resisting lateral loads.

In the late 1970's, ACI Committee 318 made a concentrated effort to provide Code directions that would incorporate flat slabs and plates as part of the lateral load resisting system. Certain factions within this committee recommended the extension of the use of the Equivalent Frame Method (EFM) (ACI-13.7) [3], which was developed for gravity loads, thus creating a unified gravity and lateral load design approach. Other members of the committee recognized the complex nature of this approach. They pointed out that the EFM was specifically developed for gravity loads via a limited research study of square panels. These members concluded that it would be imprudent to use the EFM in the design for lateral loads without additional research.

The disagreement within the Code committee prompted a recommendation to the Reinforced Concrete Research Council (RCRC) to assign the late Professor Vanderbilt the task of accumulating the available research and theoretical studies on this subject. Part of his charge was to verify if the EFM could safely analyze lateral loads.

Vanderbilt accumulated most of the information available prior to 1981 on this subject [4] and described the different approaches a design team might select. A considerable part of his report focused on the use of the EFM (a.k.a. the "transverse-torsional" method), concluding that this method would be suitable, but only with some adjustments. Prof. Vanderbilt determined adjustments were necessary in the computation of the torsional link K_t. He suggested that $\ell_2 \leq \ell_1$ be used in this computation. Disagreements persisted within the ACI Code Committee 318 (for the 1983 Code) and the proposal was tabled.

At this time, the author of this paper developed an "effective width" design model [2] that was extracted from the various papers and meager research information reviewed in [4] and was modified based on engineering judgment. This code proposal was tabled due to a lack of experimental verification. Due to the committee's lack of consensus regarding this issue, the ACI 318-83 Code [5] is vague (see Commentary [6] Section 13.3.1.2). It became obvious that more experiments were necessary. Meanwhile, established practices continued.

In flat slabs, a lack of proper detailing in the joint between the column and the slab can cause a considerable loss of stiffness. ACI 318 (section 13.3) specifies the portion of the unbalanced moment (at square supports it is equal to 60%) to be transferred by flexure, via concentration of reinforcement within a narrow band, "column head," which equals a width of (C_2 + 3h), where C_2 is the transverse width of the support and h is the slab thickness. The remainder of the unbalanced moment is transferred by the eccentricity of shear about the centroid of the critical section (ACI 318, section 11.12). This empirical method does not accurately describe the transfer of unbalanced moments, but is assumed to generally provide conservative results. Unfortunately, it is too rigid for use in workable design. A description of the evolution of this method is in reference [7], along with recommendations that allow for the more flexible provisions necessary for workable application.

The review of the state-of-the-art assembled by Prof. Vanderbilt indicated to the critical analyst that no explicit rational directions could be provided without additional pertinent research. Most attempts to provide direction for both the "transverse-torsional" and the "effective width" design models were based on the elastic plate theory -- modified and "ironed out" to agree with the meager test results available. In the attempt to correlate theory and tests, an important parameter, the ability of the connection between the floor member and the support to develop the predicted unbalanced moment and shears, was generally not considered. Another often neglected parameter was the availability of redundancy, which allows for the redistribution of and better utilization of available capacity elsewhere in the structure. On the flip side of the coin, construction procedures and loads could reduce available stiffness more than anticipated by the presence of the service loads. Tests of three-dimensional models that would also review the slab-column connection (considering concentration of reinforcement within the "column head," the presence of capitals, drops, etc.) and also varied panel ℓ_2/ℓ_1, C_2/C_1 aspect ratios were necessary before any explicit rational directions could be provided. Meanwhile, the lack of testing strained the credibility of the practitioner who had to continue to design flat slab structures.

The stalemate within the 1983 Code Committee eventually initiated a "shopping list" of needed research [8] which was forwarded to RCRC. Two requests from this "shopping list" pertinent to the subject of this paper are quoted below:

A. "Re-investigate transfer of unbalanced gravity and lateral loads in flat slabs. Code provisions (section 11.12.2.3) [5] are too rigid. Ad Hoc Committee investigation (chaired by Professor James MacGregor) could use a deliberate research effort to reevaluate equation (11-40) especially with an eye to allowing for some flexibility in the portions assigned to flexure and eccentricity of shear."

B. "Study 1/3 scale 3-D models with varied aspect ratios of spans ℓ_2/ℓ_1 column sizes C_2/C_1 etc. to help formulate simple Code provisions for flat slab participation in lateral load resistance. Such a study should consider the slab-column joint. Emphasis should be on structures designed for moderate wind or seismic forces and which have a deflection index of about 500. The total reinforcing provided should be based on Code requirements to satisfy the test load. Concentration of reinforcing over support should be done by varying the spacing and size and not by increasing the total amount. Also re-examine the practicality of elastic analysis for gravity loads (such as EFM of chapter 13) in concrete structures."

Several researchers have reviewed and contributed to Item A. The Technical Committee's (ACI-ASCE 352) recent report [9] incorporated substantial improvements, allowing for a large measure of flexibility at the exterior supports. Independent studies, by this author, of available research indicated that a moderate measure of flexibility is also available at the interior supports. The flexibility in the portions assigned to flexure and eccentricity of shear is made possible when direct shear, due to gravity loads and the quantities of flexure reinforcing within the "column-head," are within prescribed lower limits. This improves ductile behavior of the slab-column joint. ACI Code Committee 318 reviewed and implemented above recommendations in the recently published 1995 Code.

The investigation of Item B was assigned by RCRC to Prof. Jack P. Moehle. His research [1] is possibly the first effort on this subject that allows a "rational" approach to resolving the effectiveness of flat plates in resisting lateral loads. Several papers [10, 11], describing the results of this research and a proposed methodology for the lateral load analysis of flat-plates, have been reviewed by RCRC (verification of the proposed design methodology for lateral loads that resulted from this research is provided later in this paper). The research also reviewed gravity loads Code procedures and concluded [10] that, "Neither the Direct Design Method (DDM) or the Equivalent Frame Method (EFM) of ACI 318-83 accurately reproduce the moment fields of the slab under service gravity loads; however, either could have been used in design to produce proportions and reinforcement similar to those used in the slab. Hence, either would have produced acceptable service load performance." The research concluded that flat plates have a large capacity to redistribute both gravity and lateral moments. There has been observation of this redistribution in many cases of actual construction. This should point the way toward the simplification of existing Code provisions (EFM) for gravity loads.

Research / Design Significance

Rational methodologies to estimate the contribution of slabs to the stiffness of the 3-D structure are presented. These methodologies consider the stiffness degradation of flat plates caused by construction and the various lateral load levels usually of interest to the designer of the structure (serviceability, strength design, limit state). The process of correlating the most accurate methodology to actual construction is underway -- with early testing based on actual weather conditions at low and moderate load levels indicating satisfactory results. A more accurate evaluation, by the designer, of the dynamic properties of structures in which flat slabs/plates are integral parts is now possible. This will result in better estimations of: the lateral loads induced on the structure; the distribution of such loads to the individual members; and the structure's sway at the various load levels considered.

Rational Methodologies for "Effective Width"

The concept of "effective width" representing the stiffness of an "equivalent beam" has been thoroughly reviewed in the literature and is summarized in [4]. The effective width factor, α, is obtained from the requirement that the stiffness of a beam of equivalent width, $\alpha \ell_2$, will equal the stiffness of the full width, ℓ_2, of the flat plate panel. The effective width accounts for the behavior of the slab which is not fully effective across its transverse width.

Factors affecting the effective width are numerous. Elastic plate analysis can identify those factors which are more dominant than others. However, for a non-homogenous material, such as concrete, a review of actual field conditions is necessary to supplement theory. Therefore, construction loads and procedures must be considered, along with the concentration of reinforcing over the supports: C_2/C_1 and ℓ_2/ℓ_1 aspect ratios; the development of cracks caused by shrinkage, by restraint of stiff supports or by loads; the capacity of the joints to develop unbalanced moments and to redistribute excessive demands; etc. Above all, prudent methodologies must describe the degradation of the stiffness of the slabs due to the level of the lateral loads.

Each of the three methodologies described in this paper is put to a sensitivity review to match the UCB test results [1]. The UCB test encompasses a variety of parameters such as aspect ratios C_2/C_1 and ℓ_2/ℓ_1, gravity loads and construction procedure influences. While other parameters have yet to be explored (such as the effects of penetrations, unequal column spans in a unilateral direction, plan offset of columns, etc.) the UCB tests do provide an initial means to develop a "rational" approach to verifying the contribution of flat plates to lateral stiffness in actual construction. The three design methodologies reviewed in this paper are designated as follows:

- Methodology "TWR" (Extracted from the various papers and meager research information reviewed by Vanderbilt in Reference [4]. This method was modified based on engineering judgment.)
- Methodology "JSG" (One of several sensitivity studies of Methodology "TWR" in which a few parameters have been altered.)
- Methodology "HWNG" (Developed by UCB researchers [11].)

Methodology "TWR"

Description of Proposed "Effective Width" Rules

In the late 1970's (as part of the effort for Code input described earlier in this paper -- see "Background"), this author assembled a design methodology to obtain the effective slab width which correlates to the acceptable drift limit (at design load levels) of about $h_s/400$ to $h_s/500$. An abstract of this design methodology was published [2] with a footnote disclaimer that the results must be verified by tests in progress at the time [1]. This methodology is duplicated below, with some minor adjustments.

The effective width at the center line of an interior slab-column joint, $\alpha\ell_2$, is computed as follows:

$$\alpha\ell_2 = 0.3\ell_1 + C_1\,(\ell_2/\ell_1) + (C_2 - C_1)/2 \qquad \text{Eq. (1)}$$

Eq. (1) allows the recognition of non-square columns and panels and the transition from two-way to one-way slab action as support width C_2 approaches panel width ℓ_2. In Eq. (1) the terms are defined as in Chapter 13 of the ACI Code, except that interior supports ℓ_1 is taken as the average of the two spans parallel to the direction of the lateral load -- one in front of and one in back of the slab-column joint. The effective width of the slab supported by two adjacent columns is then taken to equal the average of the values at the supports, and has the limits:

$$0.2\,\ell_2 \le \alpha\ell_2 \le 0.5\ell_2 \qquad \text{Eq. (2)}$$

To evaluate $\alpha\ell_2$ at edge columns, with the edge of the slab parallel to the direction of the lateral load, the following procedure was used:

> ℓ_2 is assumed to equal the transverse centerline distance to the column in the adjacent interior panel. $\alpha\ell_2$ is computed as if the edge column is an interior column having ℓ_2 thus obtained. Adjustments are then made by multiplying $\alpha\ell_2$ by Eq. (1) by $(\ell_3 + (\ell_2/2))/\ell_2$ where ℓ_3 equals the distance between the column centerline and the parallel edge of the slab.

Reference [2] indicated that Eq. (1) appears to provide "rational" estimates of stiffness for two-way slabs to be used in a second-order analysis in structures restricted by the drift and stability boundaries established in the reference. "Rational" stiffness is defined here as the stiffness of the structural member at, or close to, the design (service) load level. The drift and stability boundaries recommended [2] were developed to provide serviceable structures for lateral wind loads and to minimize adverse P- Δ effects, should the initial estimate of member stiffness by the designer be significantly erroneous. These requirements dictated that the structure being designed have:

1. Final (second order) story deflection index $h_s / \Delta_f \geq 400$ and total structure deflection index $\Sigma h_s / \Sigma \Delta_f \geq 500$ (with loads magnified due to P-Δ effects).
2. Small stability index $Q = (\Sigma P) \Delta_i / Hh_s \leq 0.15$ using a rational stiffness estimate.
3. Provision for ductility and redundancy in the structure.

[Service loads are used in the above: Δ_f = final story deflection (including P-Δ effects); Δ_i = initial 1st order story deflection; h_s = story height; H = lateral load accumulated down to level i; ΣP = total gravity loads for levels above and including level i.]

Reference [2] indicates that the small stability index Q was selected in order to assure no larger than 10% increase in lateral shears and moments, even if erroneous stiffness assumptions (sufficient to increase the computed sway by about 50%) are selected by the designer. Since the estimate of stiffness was in doubt, this approach minimized the penalty erroneous assumptions might have caused. Notice that service loads are used in both the sway and stability computations.

Proper detailing of reinforcing is also necessary to develop the moments and to allow redistribution where necessary. Concentration of reinforcement to develop the unbalanced moment within the "column head" (C_2 + 3h) should be allocated so that the ratio of top to bottom reinforcement is about 2 to 1 when reversal of moment occurs on both sides (windward and leeward) of the column. This reinforcement is sized to be developed within column dimension C_1 and should not exceed $0.375\rho_b$ in order to allow for ductile behavior and redistribution of excessive moments without spalling of the concrete [18]. Concentrating reinforcement within the "column head" was deemed necessary to improve all aspects of slab behavior and maintain the stiffness of the slabs.

Eq. (1) was intended to account for a "normal" amount of accidental cracks developed during the construction process (in addition to the design loads), some yielding of the reinforcing over supports and some bar slippage (all are usually observed in average construction). Reference [2] suggests that this design methodology is applicable for structures in mild climate zones where lateral wind or seismic forces are low so that the structure is generally behaving elastically. In moderate seismic zones, about 75% of the values given by Eqs. (1) and (2) were recommended.

A footnote disclaimer alongside equation (1) in reference [2] reads:

> "Laboratory testing now (May 1986) underway at the University of California at Berkeley to study vertical and lateral load resistance of flat-plate construction should disclaim or verify the accuracy of this equation."

The results of this test at UCB [1] are used to verify the various design methodologies reviewed in this paper.

At the time "TWR" was developed (the late 1970's) it was compared to a methodology described by Kahn and Sbarounis [12] and was found to produce slightly more flexible structures. It has been used since the early 1980's in the design of over two hundred structures.

The use of any methodology, including "TWR", requires several "judgment" calls to allow it to be used in actual construction. Slabs in structures are penetrated by many sleeves and slots. Often, more than half the supports are alongside slab edges. The designer must use "judgment" in further reducing stiffness based on the location of the slab edges or the size and proximity to supports of the mechanical penetrations. The design "judgment" necessary to account for penetrations and edge conditions cannot easily be described in Code language. Clearly, additional research on this subject is necessary.

This author associated the reduction in joint confinement of an edge column (where the lateral moment is parallel to the edge) with a 20% reduction to the effective width of the slab. This value was not based on laboratory studies, but was selected as representing one side lost which contributed torsional capacity to transfer moments (for square interior supports with both transverse faces intact, 40% of the unbalanced moments are Code prescribed to be transferred by other means than by direct flexure). Exterior columns with the edge of the slab perpendicular to the direction of moment also suffer from reduction in joint confinement and will elongate or shorten due to axial chord action so that a portion of the captive moment will be distributed. Therefore, a similar reduction (20%) was used for these columns as well. The effective width of slabs at corner columns (with two faces devoid of slabs) was reduced to 64% (0.8 x 0.8) of the effective width computed by Eq. (1).

Methodology "TWR", as described above, was extensively reviewed for compliance with the results of the UCB [1] test. The comparison of measured drift results to computed results (at load levels producing drifts of $h_s/400$) indicated a need for a small adjustment to Eq. (1) when the flat plate is very thin and $d/h < 0.9$.

An earlier parametric study by this author [15] evaluated the parameters influencing the effective moment of inertia, I_e, by ACI Code Equation (9-7) for slabs and beams supporting gravity loads. This parametric review was made for the parabolic moment envelope (the results of gravity loads acting alone) and not for the combined moment envelope of gravity and lateral loads (a thorough research of this item is still needed). However, section 13.3.1.2 of ACI Commentary [6] indicates that reasonable estimates of stiffness of beams ($E\, I_e$) for lateral loads may also be obtained by using Code equation (9-7) to compute I_e.

Reference [15] suggests simplified equations to compute I_e directly:

When $M_a/M_{cr} > 1.6$:

$$I_e / I_g = 0.1(M_a/M_{cr})K_e \geq 0.35K_e \qquad \text{Eq. (3)}$$

$$\text{Where } K_e = (d/0.9h)\,\{1/[0.4 + (1.4\,M_a/\varnothing M_n)\,(fy/100{,}000)]\} \qquad \text{Eq. (4)}$$

[In Eq. (4) the value enclosed in the bow brackets { } is equal approximately to unity when fy = 60,000 psi., therefore, in this common case, $K_e \approx d/0.9h$].

It is thus recognized that I_e / I_g is, in part, dependent on the ratio d/0.9h. When this value is reduced, as in thin slab members, the stiffness of the slab is also reduced. Therefore, it is necessary to make adjustments to $\alpha\ell_2$ of thin slabs by multiplying $\alpha\ell_2$ in Eq. (1) by d/0.9h. This small adjustment brings measured and computed drifts in close conformity to each other.

Eq. (1) at interior supports could now be expanded to also include edge, exterior and corner supports along side adjustments for thin slabs as follows:

$$\alpha\ell_2 = [0.3\,\ell_1 + C_1\,(\ell_2/\ell_1) + (C_2 - C_1)/2]\,(d/0.9h)\,(K_{FP}) \qquad \text{Eq. (5)}$$

where K_{FP} = 1.0 at interior supports
= 0.8 at exterior and edge supports
= 0.6 at corner supports

The effective width of edge supports (with the edge parallel to the direction of the lateral load) requires a final step to adjust the results of Eq. (5) by the factor $(\ell_3 + \ell_2/2)/\ell_2$ described earlier.

Eq. (5) describes the effective width of slabs which have degraded in stiffness by lateral loads, causing a critical story sway of about $h_s/400$.

The results comparing predicted behavior of the UCB test by the above described procedure for a range of story drift indexes are displayed later in this paper, under subheading "TWR" (Tables D, E, F, and G).

Methodology "JSG"
Description of Proposed "Effective Width" Rules

Methodology "JSG" is an offshoot of Methodology "TWR" and is one of many sensitivity reviews whose purpose is to evaluate how the various parameters influence the effective width of flat plates and to ascertain if improvements in the accuracy of predicting behavior can be realized. In this methodology all computational procedures are identical to the procedures used in Methodology "TWR," except Eq. (6) replaces Eq. (5).

In Eq. (6) the average of the clear spans ℓ_{1n} (rather than c/c spans) is used and ℓ_2 is set to be smaller than or equal to ℓ_1 (the latter adjustment is incorporated in order to examine if Vanderbilt's recommended adjustment for EFM [4] has validity here). Smaller effective width values were the result of these adjustments which, in turn, produced somewhat larger drift estimates.

$$\alpha\ell_2 = [0.3\,\ell_{1n} + C_1\,(x) + (C_2 - C_1)/2]\,d/0.9h\,(K_{FP}) \qquad \text{Eq. (6)}$$

where $x = \ell_2/\ell_1 \leq 1.0$

The results found by comparing the predicted behavior of this methodology to the measured results of UCB tests [1] are displayed later in this paper, under subheading "JSG" (see Tables D, E, F, and G).

Methodology "HWNG"

Description of Proposed "Effective Width" Rules

This methodology was developed by the researchers of the UCB tests. It is described in depth in Reference [11] and is summarized below.

For interior supports and edge connections with bending perpendicular to edge.

$$\alpha\ell_2 = (2C_1 + \ell_1/3)\beta \qquad \text{Eq. (7)}$$

For edge supports with bending parallel to the edge.

$$\alpha\ell_2 = (C_1 + \ell_1/6)\beta \qquad \text{Eq. (8)}$$

where $\beta = 5C_1/\ell_1 - 0.2\ (LL/40 - 1) \geq {}^1/_3$ Eq. (9)

or, approximately, $\beta = 4\ (C_1/\ell_1) \geq {}^1/_3$ Eq. (10)

(LL = live load or construction loads and β accounts for loss of stiffness under loads).

In the following sensitivity review the approximation for β given in Eq. (10) is used. No other adjustments (similar to K_{FP} for "TWR") are made at edge and corner columns, as none were recommended by the researchers and the results of initial studies indicated that larger discrepancies between measured and computed deflections will occur when such adjustments are employed.

The results found by comparing the predicted behavior of this methodology to the measured results of the UCB tests [1] are displayed later in this paper, under subheading "HWNG" (see Tables D, E, F and G). It was observed that Methodology "HWNG" more closely described the effective width of the slab at a load level, causing a critical story sway of about $h_s/200$.

The UCB Test

Description and Adjustments Needed

Figures 1, 2a and 2b indicate the variety in geometry. Tables 1 and 2 indicate the chronology of the Flat Plate tests and their results at UCB [1]. This information is duplicated from the UCB report with the generous permission of Professor Moehle.

The purposes of this test, to review the Code's gravity load requirements and to estimate the lateral stiffness of flat plates, were further extended to see if reduced Code requirements would provide satisfactory behavior. The placement of reinforcing at the south half of the test model followed ACI Code minimum requirements, while liberties were taken to cause a large redistribution of moments in the north half by reducing the amount of negative steel and increasing, where needed, the positive steel reinforcement. This test indeed verified that large redistribution is possible for both gravity and lateral loads. However, the overall reduction in total reinforcement, especially in top steel, (see Fig. 2a) did reduce the stiffness of the north half of the test slab.

In order to develop design criteria for the effective width of a flat slab, the effects of the reduced stiffness of the north half had to be considered. This was accomplished by dividing the measured slab-column connection stiffness in the north half by the measured stiffness of a similar connection in the south half of the test model. The average of two tests in each direction were used to estimate this stiffness ratio, R1 (NS400 with NS200 in the N-S direction and EW400 with EW200 in the E-W direction). The results are listed in Table A. The drift index of 400 is in close proximity to the recommended drift index of 500 in reference [2] for wind design. The drift index of 200 is approximately the recommended index for seismic in several national Building Codes. Thus, these tests were selected (over others listed in Table 1) to estimate the reduction in stiffness in the north half.

Gravity loads of the single flat plate level tested were quite small and could not provide the stiffening effects larger gravity loads provide to columns in high rise structures. The test model columns were heavily reinforced and the cracked moment of inertia was used in the analysis [1]. Table B indicates the ratio of $R2 = I_{cr}/I_g$ of the supporting columns for the test model.

The stiffness adjustment, R3 (see Table C), for the effective panel width in the North half of the test model, is obtained by averaging R1 values (slab-column connection stiffness ratio from Table A) at each end of the panel.

Comparison of "Effective Width" Rules to Test Data

First order analysis (P-Δ is negligible) by SAP90 software [16] was utilized to verify the accuracy of the design methodologies tested. Rigid joints were assumed in the analysis. The actual constructed sizes (such as 3.3" average slab thickness) and measured modulus of elasticity as presented in [1] were used. The corrections to stiffness based on the softer north half were applied by multiplying the modulus of elasticity, E, of the members in the north half by R1, R2 and R3.

The stiffness of the test model in the E-W direction was influenced similarly by the softer north-half, regardless of the direction of the load. However, in the N-S direction, when the load action was toward the north, the influence was greater because the limited top steel at the north building edge (in tension for this loading direction) provided minimal stiffness. For that reason evaluating the results in the N-S direction should lean more heavily on tests with the load acting south. In the E-W direction either test (load east or west) can be used.

The results of the three design methodologies described in this paper are compared to the test results at UCB [1] (see tables D, E and F). The measured column shears were considered to provide the best means to gauge the accuracy of the design methodologies. In Table D, column shears from NS400 (total shear of 9100 lb in the southerly direction resulting in a drift of 0.12"), and from EW400 (total shear of 11,470 lb in the westerly direction, resulting in a similar drift) are compared to the predicted computer analysis of the three design methodologies and are listed in reverse order of compliance with the UCB test results.

The computed columns shears for each of the above described methodologies are tabulated alongside the test results. The non-square supports (Tables D-2 and D-4) were separated from the square supports (Tables D-1 and D-3) to better determine the effect of member geometry. Standard deviations (assuming test shear results as a base 100%) are also provided. Finally, drift predictions by the three methods and standard deviation for shear distribution for the total structure are shown in Table D-5.

Table-E for drift index of 200 and Table-F for drift index of 800 are similarly provided <u>without</u> further adjustments to member stiffness (i.e. stiffness of members remain unchanged in each of the Tables D, E, and F).

Discussion of Results

The purpose of this review is to establish a "rational" design methodology to estimate the stiffness contribution of flat plates. Many designers will provide a structure with sufficient stiffness so that for the common service design load level (50 to 100 year wind force or mild to moderate seismic forces) a sway index of 400 (or better) for the critical story is realized. This is the load level the design methodologies reviewed should be tailored to satisfy. The results of the comparison of these three methodologies to the UCB test at a sway index of 400 are tabulated in Table D. Table E tabulates the comparison between the methodologies at larger drifts ($h_s/200$), which are more likely to occur at moderate to more severe seismic design loads. Table F provides the comparison at lower load levels

(6 to 10 year return) for which serviceability (comfort, perception to motion) are being reviewed and which may cause sways not to exceed approximately (h_s/800).

The Code's Commentary [6] (Section 13.3.1.2) suggests that a conservative assumption of drift is appropriate for structures of unbraced frames. However, in structures having dual-systems (shearwalls and frames) a more accurate prediction of slab stiffness is desirable so that the frame members are properly proportioned to resist the lateral forces and moments their relative stiffness will attract. Since the dual-system is more dominant in structures where drift limits are of concern, this review will attempt to develop a methodology which accurately predicts the sway.

Tables D through F have been prepared using Eqs. (2) and (5) for methodology "TWR"; Eqs. (2) and (6) for "JSG"; Eqs. (7), (8) and (10) for "HWNG." These equations have not been adjusted, as yet, for the different levels of stiffness anticipated at each load level of design concern indicated above. The ratio of computed to measured drifts in Tables D through F allows us to determine a "stiffness degradation" factor, K_d, so that a single unified methodology can provide a measure of the flat plate contribution to stiffness at any load level of design concern (see Table G).

DRIFT PREDICTIONS:

The ability for methodology "TWR" to predict drifts is sufficiently accurate for design purposes as it "straddles" the measured test results. The computed/measured ratio (see Table D) is 92% and 108% in the N-S and E-W directions respectively for drift index h_s/400. In most residential structures (in which flat slabs are paramount), rectangularity of panels is random and is shared in both directions. Therefore, it is likely that a small over-estimation or under-estimation, based on the panels rectangularity, will be balanced in the average structure. This methodology is, therefore, verified to provide "rational" drift values at load levels appropriate for wind induced sways (h_s/400), without additional adjustments to stiffness (K_d = 1.0).

Adjustments for stiffness degradation, K_d = 0.8, straddles the computed/measured drift (see Table E) for Methodology "TWR" at a drift index of 200 and will provide sufficiently accurate drift results at this design level. For a drift index of 800 it is necessary to increase the effective width to 1.1 ($\alpha \ell_2$) (therefore, K_d = 1.1) in order to more accurately estimate the available stiffness. Similar corrections are also adequate for Methodology "JSG". Methodology "HWNG" provides overly conservative drift estimates at a drift index of 400 and is more suitable to estimate drifts at a drift index of 200 (for which K_d = 1.0). Corrective ratios (not provided by the researchers) of K_d = 1.33 and K_d = 1.45 are appropriate to increase stiffness at drift indexes of 400 and 800 respectively. The increased stiffness can be realized by multiplying β obtained from Eq. (9) or (10) by K_d.

When the various corrective measures (K_d) indicated above are applied to the computed effective widths, the ratios of computed to measured drifts indicated in Tables D through F will be adjusted (neglecting the small column contribution to drift) approximately as shown in Table G.

TABLE G
Computed Drift / Measured Drift - Using Adjustment K_d

Drift Index	Methodology "HWNG"			Methodology "JSG"			Methodology "TWR"		
	N-S	E-W	Avg.	N-S	E-W	Avg.	N-S	E-W	Avg.
$h_e/400$	94%	106%	100%	103%	111%	107%	92%	108%	100%
	$K_d = 1.33$			$K_d = 1.0$			$K_d = 1.0$		
$h_e/200$	105%	110%	108%	106%	109%	108%	96%	106%	101%
	$K_d = 1.0$			$K_d = 0.8$			$K_d = 0.8$		
$h_e/800$	93%	107%	100%	100%	111%	106%	91%	107%	99%
	$K_d = 1.45$			$K_d = 1.1$			$K_d = 1.1$		

With these K_d adjustments, more accurate estimates of sways at various load levels may be realized from any of the three methodologies reviewed. Methodology "TWR", however, also provides (as will be shown below) the greatest accuracy in distribution of the lateral forces to the individual members. The adjusted stiffness of Methodology "TWR" are superimposed on Figs. 3 and 4 (Figs. 6.9 and 6.10 in Ref.1 - these figures compare the accuracy of the various methodologies described in Ref.1).

SHEAR DISTRIBUTION PREDICTIONS:

Tables D through F indicate that design methodology "TWR" provides the most accurate prediction of shear distribution in square and non-square columns at all drift indexes. For example: At drift index of 400, standard deviation for shear distribution into all the columns is 6.6% in the North-South direction and 7.1 % in the East-West direction (see Table D-5).

Results for methodology "JSG" indicate somewhat less accuracy in predicting the distribution of lateral shears to the supports. Vanderbilt's recommendation to limit $\ell_2 \leq \ell_1$ for the torsional link of the "EFM" Model did not improve the accuracy of shear distribution for the "effective width" Model and is, therefore, not warranted here. For example: At drift index of 400, standard deviation of 6.8% in the N-S and 7.7% in the E-W (see Table D-5).

Methodology "HWNG" totally ignores parameter ℓ_2 and the rectangularity of support in computations for $\alpha \ell_2$. It is shown to have reduced accuracy in distributing the lateral shears to the various members. For example: At drift index of 400, standard deviation of 10.8% in the N-S and 10.1 % in the E-W (see Table D-5).

"Rational" Design Recommendations For Structures With Flat Slabs

Structures must be investigated at different stages for different design parameters. In the design for wind, serviceability for non-structural elements (such as partitions and cladding) must be reviewed at design level forces. The threshold of non-structural damage to partitions is estimated to occur at a (service load level) drift index of about 400. If not exceeded, this critical story drift index will also minimize an adverse increase in P-Δ effects, should the initial estimate of stiffness by the designer be significantly erroneous. This will influence the design for the whole structure (for design load levels) to be at about 500 or more to keep the more critical levels at 400. In addition, for certain lightweight and slender structures [17] serviceability to minimize perception of motion must be also reviewed. In this case, stiffness dependence is only part of the issue - perception of motion is a function of mass, damping, a host of other parameters, and, to a lesser extent, of stiffness. This review for serviceability is made for a more frequent occurrence of (even though lower) wind loads. At this load level, the structure's period is shorter and its damping is lower. One may anticipate a drift index of the order between 2000 and 800 for this review. For seismic design a more inelastic behavior is anticipated. However, Code drift limits are more liberal (200 $\pm$).

The dynamic properties of the structure, its damping and periods, will influence the design loads for both wind and seismic design. Wind tunnel testing will predict larger wind loading for larger period buildings because wind gust energy is concentrated at high periods. Seismic loads will generally be reduced for tall buildings having periods 2 seconds or larger, once the structure is forced to enter a post-yield state. Additional cracking at the post-yield state will increase the damping and elongate the periods of the structure. Both these functions will reduce the lateral loads on the structure. Therefore, it is proper to obtain base shear for seismic loads in tall structures using the partial secant stiffness at a drift index of 400 which will provide a threshold of initial yielding in the structure.

Measured damping, at ambient load levels, of recently constructed concrete structures (with most of the non-structural elements not yet in place), varied between 1 % and $1^1/_4$%. Reference [17] proposed that for generally elastic behavior, at design wind forces, $2^1/_2$% minimum damping could be anticipated. For serviceability at intermediate wind forces, to estimate perception to motion, $1^1/_2$ - 2% damping range should be assumed. For seismic events, anticipating larger excursions into the inelastic range, 5% damping has usually been assumed for design purposes by the profession with much larger 10 - 20% and more anticipated just prior to collapse.

Setting design criteria for $h_s/400$, $h_s/200$ and $h_s/800$ for non-homogenous material such as concrete is a complex proposition. The stiffness of the structure and its damping at any of these drift levels are also dependent on previous events. A review with pre-assigned anticipated degradation levels, to establish periods and range of damping for each design parameter, should encompass any eventual probability of design requirements. For elastic (wind) design, a limit state having lower member stiffness of about 70% of the "rational" stiffness proposed at service loads is now required by the 1995 ACI Code. For seismic an additional level of stiffness degradation must be reviewed (the stiffness just prior to a loss of 20% $\pm$ of ultimate capacity) in order to evaluate collapse-prevention requirements which will test the structure's strength and ductility to the utmost. The UCB tests indicated (Table 2) that such a loss of strength did not occur until the structure exceeded 2.0% drift. Proper design for life-safety should consider limiting drift, for unreduced base shear loads, to about 1.0% of the structure's height. At this state, the stiffness of the structure may degrade to approximately 50% of its stiffness at $h_s/400$ (see Table 2).

Ambient measurements of a structure's periods and damping can be readily obtained from the completed structure. Preferably, this is done before most of the non-structural elements are added. At such time, the structure is 6 - 18 months old (depending on its size). It has undergone several wind storms, as well as experiencing the unkind influences of the construction procedure and construction loads. A large percentage of shrinkage and creep is also present. It is difficult to use ambient measurements to predict the dynamic properties of the structure at later dates at load levels which require design for serviceability and strength. Measurements must be taken at these load levels, or laboratory testing (such as provided, on a limited scale, by the test at UCB [1]) must be performed. Ambient measurements do provide, however, a more reliable starting point from which to project and verify the design assumptions and, if they are found to be amiss, to provide the artificial means (for example, dampers) to improve future behavior [17].

Ambient measurements of "weathered" (old) structures taken at a low level of lateral loads will depend heavily on the past historical events influencing the structure. In most cases, such measurements of the dynamic properties will indicate longer periods and larger damping values than in newly constructed structures. A UCB [1] recorded "ambient" reading is taken (see Table 2, LAT1) at a sway index of 800 in the N-S direction. The initial stiffness recorded at this level is 177% of the average NS400 stiffness. In reference to this value, it might be reasonable to assume that "ambient" readings of frame structures will be, on average, between 1.5 to 2 times the frame stiffness at design load level - resulting in a sway index of 400. It can thus be anticipated that the predicted period of slab-column frames at a drift index of 400 will be about 20-40% longer than the measured period at "ambient." It is expected that non-structural elements, if present, will shorten the ambient measurements and this must somehow be discounted. Conversely, an older building which has serviced many wind storms will have reduced stiffness (longer period) at ambient, closer to the period at the design level. The engineer's review of the available facts must be included in the process of evaluation.

This summary points out the need for additional information, which can only be possible if actual. structures are monitored. There is a need to measure both newly constructed and "weathered" structures in order to verify the methodologies used in their design and to determine the affect non-structural elements provide. A few (though, not enough) measurements toward this end have already been taken [20]. The process of verification of the proposed design methodology has also been started with encouraging initial results. See Appendix B for preliminary correlation between the methodology and the field measurements of the dynamic properties of newly constructed structures utilizing ambient (small wind) forces and the largest wind storm encountered to date.

For Methodology "TWR" -- tentatively, based on UCB tests (until additional tests involving actual structures at higher load levels, not yet encountered, can verify or refine such values), K_d, a factor estimating effective width degradation in stiffness at various load levels (assumed equal to unity at $h_s/400$), may be taken to equal:

At "Ambient" $1.5 \leq K_d{}^* \leq 2.0$

$h_s/800$ $K_d = 1.1$

$h_s/400$ $K_d = 1.0$

$h_s/200$ $K_d = 0.8$

$h_s/100$ $K_d = 0.5$ (for unreduced base shear loads at 1.0% drift)

K_d values shown above exclude participation of non-structural elements.

* use $K_d = 2.0$ for "young" structures and $K_d = 1.5$ for older structures

Equations (5) and (2) for Methodology "TWR" can now be adjusted to also relate to the various load levels:

$$\alpha\ell_2 = K_d\,[0.3\ell_1 + C_1\,(\ell_2/\ell_1) + (C_2 - C_1)/2]\,(d/0.9h)\,(K_{FP}) \qquad \text{Eq. (11)}$$

with limits: $$(0.2)(K_d)(K_{FP})\,\ell_2 \leq \alpha\ell_2 \leq (0.5)(K_d)(K_{FP})\,\ell_2 \qquad \text{Eq. (12)}$$

Where K_d is a function of the load level to be reviewed and the targeted drift index. It is assumed that the members involved will be reinforced adequately in order to provide the necessary strength and ductility to the structure. (Consult the section - Methodology "TWR" Description of Proposed Effective Width Rules - for adjustments required at edge, exterior and corner supports).

Methodology "JSG " requires the same K_d adjustments as described directly above.

Methodology "HWNG" requires an adjustment to reflect that $K_d = 1.0$ for sway of $h_s/200$ (see Table G).

Other structural elements (beams, shear walls) degrade in a somewhat different fashion. Reference [2] discusses this author's design assumptions for columns, beams and shearwalls at service loads. (A summary is presented in Appendix A).

Isolated beams (without flanges) do not have the ability flat plates (or, to a lesser extent, flanged beams) have to mobilize reinforcing and concrete from areas further away from the supports. Table 2 indicates that for flat plates some strength gain was observed in the UCB test, even at extreme drift ratios of $h_s/50$ (2% drift). Ductility requirements for flat plates, when part of a dual system, which are designed for seismic loads not to exceed (say) 1 % drift are therefore much smaller [19] than for shear-walls at this drift limit. Sufficient ductility requirements can be provided by keeping slab reinforcing ratios over supports moderate ($\rho \leq 0.375\rho_b$) [18] to assure redistribution capabilities without spalling of the concrete and by keeping the punching shear stress due to gravity loads at low levels (not to exceed $1.5\sqrt{f'_c}$ [19]). Such requirements are needed to allow unbalanced moments resisting the lateral loads to develop between slabs and supports, while protecting the gravity load carrying capacity at the slab-column joint.

Conclusions

The flat plate/slab system is the most common system found in residential and other structures. It is tough (if not stiff). The slab-column joint, because of its large plan size to height ratio, does not generally require special reinforcing within the joint. It is an excellent diaphragm engaging all supports to provide added redundancy to the structure. It does contribute stiffness to the structure and therefore should not be ignored. Present Codes allow participation of flat slabs/plates to resist wind forces and base shear in moderate seismic zones. This system works best when coupled in a dual system with at least some shear-walls. In high seismicity zones ignoring the stiffness of the slab could cause harm, as unforeseen torsional effects or under-strength conditions at the slab-column joint may precipitate local failures. This author encourages the participation of this system and the consideration of its stiffness in all seismic zones. Methodologies which are simple to use by the design profession will encourage designers to improve the economy of construction by also incorporating the stiffness of the slab in the structural system. To provide this means - three simplified methodologies to estimate effective width, $\alpha\ell_2$, of slabs in slab-column frames were reviewed in this paper.

These three methodologies were calibrated to match the measured deflections of a 3-D flat slab-column frame from UCB [1] experimental data. The test considered the affects of connection and panel geometry, as well as the cracking caused by construction, gravity, and lateral loads. Rectangular panels; square and rectangular supports; interior, exterior, edge and corner supports were all exposed to forces in orthogonal directions simulating wind and seismic events. Initial monitoring of high-rise structures provided correlation, so far, at low to moderate wind load levels.

Each one of the methodologies reviewed in this paper was found to provide an adequate estimate of the stiffness of the flat-slab if calibrated by factor K_d of Table G (used to account for stiffness degradation caused by increased loading). Methodology "TWR" Eq. [11] was found to provide the most accurate lateral load distribution into the individual elements of the 3-D frame. The simplified methodologies are "user friendly" and can be implemented into computer programs or used in manual computations.

Research Needs

The excellent research program at UCB should pave the way toward Code simplifications for gravity loads and the inclusion of a preliminary "rational" design methodology for lateral load participation of slab-column frames. Additional research should be forthcoming to: fine tune requirements; better estimate effects of penetrations; determine affects of column (plan) offsets; etc. Monitoring of existing structures and reviews to match or improve the design methodology is presently proceeding by this author's firm (as time permits), but additional formal research efforts are necessary in order to expedite and complete this process.

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Notation

d	= Effective depth of slab.
h	= Slab thickness.
h_s	= Story height.
H	= Lateral (service) load for levels above and including level i.
ΣP	= Total gravity (service) load for levels above and including level i.
Δ_i	= First order (initial) story deflection.
Δ_f	= Second order (final) story deflection (including P-Δ effects).
C_1	= Size of support in direction parallel to lateral load.
C_2	= Size of support in direction transverse to lateral load.
ℓ_1	= Length of span (c/c of supports) in direction parallel to lateral load (average of two spans at interior supports).
ℓ_{1n}	= Length of clear span in direction parallel to lateral load (average of two clear spans at interior supports).
ℓ_2	= Length of span (c/c of supports) in direction transverse to lateral load (average of two spans at interior supports).
ℓ_3	= Transverse distance between column centerline and edge of slab.
α	= "Equivalent width" factor.
$\alpha\ell_2$	= Effective width of slab at center line of support.
Q	= Stability index.
M_n	= Nominal moment strength.
M_a	= Maximum (service) moment due to lateral load.
M_{cr}	= Cracking moment (see ACI Section 9.5.2.3).
β	= Factor accounting for loss of stiffness under loads.
K_e	= Factor adjusting effective moment of inertia I_e.
K_d	= Factor considering degradation of stiffness of slabs at various lateral load levels.
K_{FP}	= Factor adjusting $\alpha\ell_2$ at edge exterior and corner supports.
	= 1.0 for interior supports.
	= 0.8 for exterior and edge supports.
	= 0.6 for corner supports.

Conversion Factors

1 inch	= 25.4 mm
1 kip	= 4.448 Kn
1 psi	= 6895.0 Pa

Appendix A

Suggested Rational Stiffness Estimate For Shearwalls / Beams / Columns / Slabs in Typical High Rise Flat Slab Structures

For service lateral loads resulting in drifts for the critical floors of about hs/400:

For shear-walls: 80% of I_g is assumed when the gravity loads prevent the wall from receiving tensile net forces, and half this value (40% of I_g) when tension occurs. (The stiffness of the wall is depending on the compressive load available. More research is needed on this item). The later reduction need be applied only to parts of the wall in tension, as compression strut action will be dominant at the base of the wall.

For columns: In tall flat slab structures, the gravity load in the columns is almost always sufficiently large to prevent net tensile forces due to combined lateral and gravity loads. The unbalanced slab moments are normally small, so the gross section of the columns can be used (except, possibly, at the top most couple of levels). Net tensile stress should be avoided. The structure is adjusted (stiffness of floor members are reduced and/or amount of flexure reinforcing curtailed) to prevent this condition.

For beams: 50% of I_g of beam sections are being used. For flanged beams a "weighted" average is being used. It is assumed that 1/3 of the beam's span is having only the stem section (tension in top of beam) and that 2/3 of the length of the beam the flange is effective (compression in top). Short coupling beams connecting parts of the shear walls are estimated based on Eq. (3) [$I_e \approx 0.1\ (M_a/M_{cr})I_g$ where M_a is the (service) flexural capacity of the beam at the face of support].

For slabs: See recommended values in this paper [$K_d = 1.0$ in Eq. (11)].

For serviceability (perception to motion) review:

Gross sections of shearwalls and beams and $K_d = 1.5$ in Eq. [11] for slabs will provide an upper bound stiffness for the structure; cracked sections for shearwalls and beams and $K_d = 1.1$ for slabs will provide a lower bound stiffness. The serviceability review considers the envelope of the dynamic properties thus obtained. 1.5% damping is used with the upper bound and 2% damping with the lower bound properties in determination of the accelerations anticipated for this range.

Appendix B

Preliminary Correlation between Methodology "TWR" and field measurements of dynamic properties of two newly constructed structures, identified as Building A and Building B (see Figs. B1 through B4) is assembled in this appendix. Additional details and information about the structures, the software, and equipment (Portable synchronized digital recorders) used to obtain the field measurements are to be found in Reference [20].

Table B1 describes efforts to verify Equivalent Width, $\alpha\ell_2$, by Eq. (11) with $K_d = 2.0$ by comparing measured and computed periods at ambient conditions. Cladding influence indicates about 10% average shortening of the fundamental periods or about 20% increase to apparent stiffness.

It is also seen in Table B1 that very good correlation between computed and measured periods (3 N-S, 3 E-W and 3 Torsion) are obtained in both buildings (except for the 3rd Torsional Mode in both buildings).

Table B2 indicates two periods of measurements that were taken from Building A during the same wind storm. During the first period (window 1), when the wind storm was at its strongest, Building A indicates a certain amount of softening. When the second set of measurements were taken (window 2), while the wind storm was milder, Building A indicates some stiffening has returned - telling us there was some recovery during this period. The periods measured during both readings fall short of the periods computed for the design (NYC wind loads). Additional high-level wind storms are necessary in order to provide an opportunity for further study, which will make final correlation of the higher design load levels possible.

TABLE B1

MEASURED VS. COMPUTED PERIODS AT AMBIENT CONDITIONS

BUILDING A: (486 feet tall)

Modes	First Reading Partially Clad	Second Reading Fully Clad	Additional Cladding Influence	Adjustment to First Reading for the Bare Frame**	Projected Measured Periods - Bare Frame	Computed* Periods - Bare Frame	Computed/ Measured Ratios
1 N-S	4.47	4.13	8%	-3%	4.6	5.2	113%
2 N-S	1.32				1.4	1.6	114%
3 N-S	0.71				0.7	0.8	114%
1 F-W	4.53	4.15	9%	0% (but adjust slightly)	4.6	4.6	100%
2 E-W	1.23				1.3	1.3	100%
3 E-W	0.64				0.7	0.7	100%
1TORSION	3.41	3.28	4%	-6%	3.6	4.1	114%
2TORSION	1.10				1.2	1.4	117%
3TORSION	0.49				0.5	0.8	160%

BUILDING B: Principal Axis is Slanted 23 Degrees. (423 feet tall)

Modes	First Reading Unclad	Second Reading Fully Clad	Cladding Influence	Adjustment to Bare Frame Not Needed	Measured Periods - Bare Frame	Computed * Periods - Bare Frame	Computed/ Measured Ratios
1 N-S	3.98	3.57	11%	0	3.98	3.93	99%
2 N-S	1.00				1.00	1.06	106%
3 N-S	0.57			NOT	0.57	0.59	104%
1 E-W	2.79	2.56	9%	0	2.79	2.63	94%
2 E-W	0.80				0.80	0.80	100%
3 E-W	0.46			NEEDED	0.46	0.47	102%
1TORSION***	3.49	?	Assume 10%	0	3.49	3.20	92%
2TORSION	1.21				1.21	1.15	95%
3TORSION	1.07				1.07	0.69	65%

Note: Building A was already partially clad at first reading, building B was bare without any partitions or cladding. Consult reference 20 for additional details.

* Computed periods were obtained using $K_d = 2.0$ for slabs [Eq. (11)] and uncracked sections for columns, walls and beams. These buildings were re-analyzed after adjustments were made to "As Built" conditions.

** Discount partial cladding in Building A based on results of cladding influence in Building B.

*** Second Torsional reading for Building B is doubtful (assume cladding decreased period by 10%).

? Questioned results require further evaluation - or new reading to verify.

TABLE B2

MONITORING BUILDING A
During Wind Storm March 4, 1993

Modes	Projected measured Periods to Bare Frame	First Reading Partially Clad Ambient	Second Reading Fully Clad Ambient	3/4/93 * 1830-1900 Window 1	3/4/93 ** 1915-1945 Window 2	Computed at Design Load *** Bare Frame
I N-S	4.6	4.47	4.13	5.03	4.83	6.56
2 N-S	1.4	1.32		1.38	1.34	1.93
3 N-S	0.7	0.71		1.14	1.12	0.93
1 E-W	4.6	4.53	4.15	4.70	4.57	5.73
2 E-W	1.3	1.23		1.26	1.22	1.58
3 E-W	0.7	0.64		0.65	0.65	0.81
1 TORSION	3.6	3.41	3.28	N/A	N/A	5.22
2 TORSION	1.2	1.10				1.74
3 TORSION	0.5	0.49				1.04

* Recorded periods from window 1 (18:30 - 19:00) March 4, 1993. Readings at nearby airport of 45mph with gusts to 58mph were recorded during this window. Earlier (not monitored) peak wind speed of 70mph was recorded.

** Somewhat reduced intensity of the windstorm was monitored for window 2 (19:15 - 19:45).

*** Period computed by ETABS [21] of the as-built structure using "rational" stiffness, $K_d = 1.0$ in Eq. (11) for flat-plates and consult Appendix A for stiffness assumptions for beams, columns and shear-walls.

Table-1

Chronology Of Tests On The Flat Plate Tested At UCB [1]

LAT1	11/04/86*	Lateral Stiffness, NS Dir., Drift 1/800, w/o Lead Weight
LAT2	11/04/86	Lateral Stiffness, EW Dir., Drift 1/800, w/o Lead Weight
LEAD	11/05/86	Gravity Load Test, 78 psf from Lead Weight
LAT3	11/06/86	Lateral Stiffness, NS Dir., Drift 1/800, w/ Lead Weight
LAT4	11/06/86	Lateral Stiffness, EW Dir., Drift 1/800, w/ Lead Weight
CONSTR	11/06/86	Construction Load Test, 55 psf (2633 Pa) Pattern Load
NS800	11/07/86	Lateral Load Test, NS Dir., Drift 1/800
EW800	11/07/86	Lateral Load Test, EW Dir., Drift 1/800
NS400	11/07/86	Lateral Load Test, NS Dir., Drift 1/400
EW400	11/07/86	Lateral Load Test, EW Dir., Drift 1/400
NS200	11/24/86	Lateral Load Test, NS Dir., Drift 1/200
EW200	11/24/86	Lateral Load Test, EW Dir., Drift 1/200
NS100	11/25/86	Lateral Load Test, NS Dir., Drift 1/100
EW100	11/25/86	Lateral Load Test, EW Dir., Drift 1/100
NS50	11/26/86	Lateral Load Test, NS Dir., Drift 1/50
EW50	11/26/86	Lateral Load Test, EW Dir., Drift 1/50
NS25	12/01/86	Lateral Load Test, NS Dir., Drift 1/25
EW25	12/01/86	Lateral Load Test, EW Dir., Drift 1/25
PF20	12/08/86	Post Failure Test, Drift 1/20

Table-2

Measured Global Lateral Stiffness At Peaks [1]

Test	Load (kip)	Deflection (in.)	Stiffness (kip/in.)	Average Stiffness (kip/in.)
LAT1	7.67 - 7.17	0.06 - 0.06	128 120	124
LAT2	5.97 - 6.33	0.06 - 0.06	100 106	103
LAT3	5.09 - 4.41	0.06 - 0.06	85 74	79
LAT4	5.90 - 5.41	0.06 - 0.06	98 90	94
NS800	4.91 - 4.52	0.06 - 0.06	82 75	79
EW800	6.27 - 6.18	0.06 - 0.06	105 103	104
NS400	9.10 - 7.66	0.12 - 0.12	76 64	70
EW400	11.47 - 10.32	0.12 - 0.12	96 86	91
NS200	15.15 - 14.48	0.24 - 0.24	63 60	61
EW200	17.93 - 18.40	0.24 - 0.24	75 76	75
NS100	23.74 - 20.16	1% 0.48 - 0.48	49 42	46
EW100	24.93 - 25.58	1% 0.48 - 0.48	52 53	53
NS50	29.36 - 29.12	2% 0.96 - 0.96	31 30	30
EW50	31.41 - 31.31	2% 0.97 - 0.97	33 32	32
NS25	36.75 - 32.49	4% 1.92 - 1.92	19 17	18
EW25	27.52 - 23.47	4% 1.44 - 1.45	19 16	18

Table - A

Stiffness Adjustments for North Half Slab-Column Connections

CONNECTION COLUMN	TEST NS400	TEST NS200	R1 N-S	TEST EW400	TEST EW200	R1 E-W
a1 a4	6750 8490	4800 6100	.79 1.00	8310 9110	4810 6510	.84 1.00
b1 b4	13470 20040	9290 12640	.70 1.00	15080 18310	11320 12610	.85 1.00
c1 c4	15080 15270	8860 11140	.91 1.00	15910 15700	11790 12730	.97 1.00
d1 d4	6390 6950	4470 5260	.89 1.00	[8990] [8750]	5340 6030	.88 1.00
a2 a3	19040 20350	15050 15740	.94 1.00	16900 20050	11580 13030	.86 1.00
b2 b3	39130 44720	[34800*] [37810]	.88 1.00	37400 46150	30450 36170	.82 1.00
c2 c3	39130 36820	29090 31210	1.00 1.00	45140 48040	32250 36920	.91 1.00
d2 d3	14180 19690	10340 12870	.75 1.00	[19990*] [18680]	10830 14150	.76 1.00

Average stiffness (kip-in/rad) values [1] are used to obtain adjustments .

* Bracketed values indicate questionable test results which were not used to determine adjustments.

Table - B

Ratio of Cracked / Gross Column Moment of Inertia

$R_2 = I_{cr} / I_g$

COLUMN SIZE	6.4 X 6.4	4.8 X 9.6	9.6 X 9.6	6.4 X 12.8
N-S DIR I_{cr}	120	75	420	240
R_2 (N-S)	$R_2 = 0.86$	$R_2 = 0.85$	$R_2 = 0.59$	$R_2 = 0.86$
E-W DIR I_{cr}	120	320	420	1000
R_2 (E-W)	$R_2 = 0.86$	$R_2 = 0.90$	$R_2 = 0.59$	$R_2 = 0.89$

Table - C

Stiffness Adjustments R_3** for Effective Panel Width in North Half of Test Slab

SLAB PANEL	N-S R_3	SLAB PANEL	E-W R_3
a4 - a3	1.0	a4 - b4	1.0
a3 - a2	$\frac{1}{2}(1.0 + .94) = .97$	b4 - c4	1.0
a2 - a1	$\frac{1}{2}(.94 + .79) = .87$	c4 - d4	1.0
b4 - b3	1.0	a3 - b3	1.0
b3 - b2	$\frac{1}{2}(1.0 + .88) = .94$	b3 - c3	1.0
b2 - b1	$\frac{1}{2}(.88 + .70) = .79$	c3 - d3	1.0
c4 - c3	1.0	a2 - b2	$\frac{1}{2}(.86 + .82) = .84$
c3 - c2	1.0	b2 - c2	$\frac{1}{2}(.82 + .91) = .87$
c2 - c1	$\frac{1}{2}(1.0 + .91) = .96$	c2 - d2	$\frac{1}{2}(.91 + .76) = .84$
d4 - d3	1.0	a1 - b1	$\frac{1}{2}(.84 + .85) = .85$
d3 - d2	$\frac{1}{2}(1.0 + .75) = .88$	b1 - c1	$\frac{1}{2}(.85 + .97) = .91$
d2 - d1	$\frac{1}{2}(.75 + .89) = .82$	c1 - d1	$\frac{1}{2}(.97 + .88) = .93$

** Estimated to equal the average of R_1 of the slab-column connections supporting the panel.

Table - D

Sensitivity Review - Comparing Shear in Columns. (h_s / 400)

Table D-1 at Square Supports N-S

ID	TEST SHEAR	HWNG400S SHEAR	JSG400S SHEAR	TWR400S SHEAR
a4	325	278 86%	277 85%	283 87%
a3	490	483 99%	477 97%	479 98%
a2	521	450 86%	446 86%	448 86%
a1	251	239 95%	238 95%	242 96%
b4	542	619 114%	551 102%	556 103%
b3	1395	1603 115%	1404 101%	1453 104%
b2	1281	1440 112%	1253 98%	1297 101%
b1	398	467 117%	414 104%	416 105%
TOTAL SHEAR	5203 100%	5579 107%	5060 97%	5174 99%
STD DEVIATION		11.2%	8.3%	7.9%

Table D-3 at Square Supports E-W

ID	TEST SHEAR	HWNG400W SHEAR	JSG400W SHEAR	TWR400W SHEAR
a4	310	247 80%	279 90%	287 93%
a3	508	486 95%	539 106%	531 105%
a2	483	409 85%	454 94%	448 93%
a1	287	209 73%	236 82%	242 84%
b4	516	478 93%	514 100%	513 99%
b3	1435	1350 94%	1417 99%	1408 98%
b2	1183	1150 97%	1202 102%	1194 101%
b1	411	419 102%	451 110%	449 109%
TOTAL SHEAR	5133 100%	4748 92%	5092 99%	5072 99%
STD DEVIATION		10.3%	9.3%	8.2%

Table D-2 at Rectangular Supports N-S

ID	TEST SHEAR	HWNG400S SHEAR	JSG400S SHEAR	TWR400S SHEAR
c4	383	379 99%	417 109%	402 105%
c3	982	914 93%	1073 109%	1408 107%
c2	1008	880 87%	1042 103%	1019 101%
c1	378	339 90%	375 99%	362 96%
d4	233	212 91%	242 104%	236 101%
d3	383	336 88%	376 98%	361 94%
d2	330	285 86%	315 95%	301 91%
d1	199	175 88%	199 100%	195 98%
TOTAL SHEAR	3896 100%	3520 90%	4039 104%	3924 101%
STD DEVIATION		11.2%	5.6%	5.5%

Table D-4 at Rectangular Supports E-W

ID	TEST SHEAR	HWNG400W SHEAR	JSG400W SHEAR	TWR400W SHEAR
c4	578	629 109%	638 110%	625 108%
c3	1593	1763 111%	1676 105%	1693 106%
c2	1605	1509 94%	1439 90%	1454 91%
c1	554	581 105%	589 106%	577 104%
d4	346	360 104%	345 100%	335 97%
d3	716	842 118%	749 105%	768 107%
d2	618	699 113%	619 100%	634 103%
d1	325	335 103%	320 98%	311 96%
TOTAL SHEAR	6335 100%	6718 106%	6375 101%	6397 101%
STD DEVIATION		10.7%	6.4%	6.3%

Table D-5

	Total Structure N-S				Total Structure E-W			
SHEAR STD-DEV	9099	9099 10.8%	9099 6.8%	9098 6.6%	11468	11466 10.1%	11467 7.7%	11469 7.1%
DEFLECT RATIO (COMP/MEAS)	0.120	0.150 125%	0.123 103%	0.110 92%	0.120	0.169 141%	0.133 111%	0.130 108%

Table - E

Sensitivity Review - Comparing Shear in Columns. (h_c / 200)

Table E-1 at Square Supports N-S

ID	TEST SHEAR	HWNG200S SHEAR	JSG200S SHEAR	TWR200S SHEAR
a4	506	463 92%	461 91%	472 93%
a3	868	804 93%	794 91%	798 92%
a2	836	749 90%	742 89%	745 89%
a1	383	398 104%	395 103%	403 105%
b4	845	1030 122%	918 109%	926 110%
b3	2438	2669 109%	2337 96%	2419 99%
b2	2244	2397 107%	2086 93%	2160 96%
b1	591	778 132%	690 117%	692 117%
TOTAL SHEAR	8711 100%	9288 107%	8423 97%	8615 99%
STD DEVIATION		16.3%	10.2%	9.7%

Table E-3 at Square Supports E-W

ID	TEST SHEAR	HWNG200W SHEAR	JSG200W SHEAR	TWR200W SHEAR
a4	527	387 73%	437 83%	448 85%
a3	817	760 93%	842 103%	830 102%
a2	771	640 83%	710 92%	701 91%
a1	423	327 77%	368 87%	378 89%
b4	784	747 95%	804 103%	802 102%
b3	2261	2111 93%	2215 98%	2202 97%
b2	2004	1797 90%	1880 94%	1867 93%
b1	670	656 98%	705 105%	702 105%
TOTAL SHEAR	8257 100%	7425 90%	7961 96%	7930 96%
STD DEVIATION		15.9%	9.3%	8.6%

Table E-2 at Rectangular Supports N-S

ID	TEST SHEAR	HWNG200S SHEAR	JSG200S SHEAR	TWR200S SHEAR
c4	663	632 95%	695 105%	670 101%
c3	1749	1521 87%	1786 102%	1744 100%
c2	1639	1466 89%	1735 102%	1697 100%
c1	478	565 118%	625 131%	603 126%
d4	412	354 86%	403 98%	393 95%
d3	650	559 86%	625 96%	602 93%
d2	554	475 86%	524 95%	502 91%
d1	321	291 91%	332 103%	324 101%
TOTAL SHEAR	6466 100%	5863 91%	6725 104%	6535 101%
STD DEVIATION		13.7%	12.4%	11.0%

Table E-4 at Rectangular Supports E-W

ID	TEST SHEAR	HWNG200W SHEAR	JSG200W SHEAR	TWR200W SHEAR
c4	945	984 104%	997 106%	976 103%
c3	2604	2756 106%	2621 101%	2647 102%
c2	2475	2360 95%	2250 91%	2273 92%
c1	910	909 100%	921 101%	902 99%
d4	491	563 115%	539 110%	523 107%
d3	948	1317 139%	1171 124%	1201 127%
d2	879	1092 124%	968 110%	991 113%
d1	455	523 115%	501 110%	486 107%
TOTAL SHEAR	9707 100%	10504 108%	9968 103%	9999 103%
STD DEVIATION		19.4%	11.9%	12.4%

Table E-5

	Total Structure N-S				Total Structure E-W			
SHEAR / STD-DEV	15177	15151 14.5%	15148 11.0%	15150 10.0%	17964	17929 17.1%	17929 10.3%	17929 10.3%
DEFLECT / RATIO (COMP/MEAS)	0.240	0.251 105%	0.204 85%	0.184 77%	0.240	0.264 110%	0.208 87%	0.203 85%

Table - F

Sensitivity Review - Comparing Shear in Columns. (h_s / 800)

Table F-1 at Square Supports N-S

ID	TEST SHEAR	HWNG800S SHEAR	JSG800S SHEAR	TWR800S SHEAR
a4	170	150 88%	150 88%	153 90%
a3	267	261 98%	258 97%	259 97%
a2	273	243 89%	241 88%	242 89%
a1	155	129 83%	128 83%	131 85%
b4	294	335 114%	298 101%	301 102%
b3	760	867 114%	759 100%	785 103%
b2	676	778 115%	678 100%	701 104%
b1	217	253 117%	224 103%	225 104%
TOTAL SHEAR	2812 100%	3016 107%	2736 97%	2797 99%
STD DEVIATION		14.5%	9.2%	8.5%

Table F-3 at Square Supports E-W

ID	TEST SHEAR	HWNG800W SHEAR	JSG800W SHEAR	TWR800W SHEAR
a4	166	136 82%	153 92%	157 95%
a3	261	266 102%	295 113%	291 111%
a2	253	224 89%	249 98%	246 97%
a1	160	114 71%	129 81%	133 83%
b4	283	262 93%	282 100%	281 99%
b3	809	740 91%	776 96%	772 95%
b2	644	630 98%	659 102%	655 102%
b1	219	230 105%	247 113%	246 112%
TOTAL SHEAR	2795 100%	2602 93%	2790 100%	2781 99%
STD DEVIATION		14.4%	10.6%	9.4%

Table F-2 at Rectangular Supports N-S

ID	TEST SHEAR	HWNG800S SHEAR	JSG800S SHEAR	TWR800S SHEAR
c4	197	205 104%	226 115%	217 110%
c3	515	494 96%	580 113%	567 110%
c2	566	476 84%	563 99%	551 97%
c1	169	183 108%	203 120%	196 116%
d4	133	115 86%	131 98%	128 96%
d3	213	182 85%	203 95%	195 92%
d2	190	154 81%	170 89%	163 86%
d1	125	94 75%	108 86%	105 84%
TOTAL SHEAR	2108 100%	1903 90%	2184 104%	2122 101%
STD DEVIATION		15.9%	12.8%	11.9%

Table F-4 at Rectangular Supports E-W

ID	TEST SHEAR	HWNG800W SHEAR	JSG800W SHEAR	TWR800W SHEAR
c4	314	345 110%	350 111%	342 109%
c3	886	966 109%	919 104%	928 105%
c2	902	827 92%	789 87%	797 88%
c1	257	319 124%	323 126%	316 123%
d4	207	197 95%	189 91%	183 88%
d3	363	462 127%	411 113%	421 116%
d2	367	383 104%	339 92%	348 95%
d1	194	183 94%	176 91%	170 88%
TOTAL SHEAR	3490 100%	3682 106%	3496 100%	3505 100%
STD DEVIATION		15.2%	14.0%	13.9%

Table F-5

	Total Structure N-S				Total Structure E-W			
SHEAR STD-DEV	4920	4919 14.7%	4920 10.8%	4919 10.0%	6285	6284 14.3%	6286 12.0%	6286 11.4%
DEFLECT RATIO (COMP/MEAS)	0.060	0.081 135%	0.066 110%	0.060 100%	0.060	0.093 155%	0.073 122%	0.071 118%

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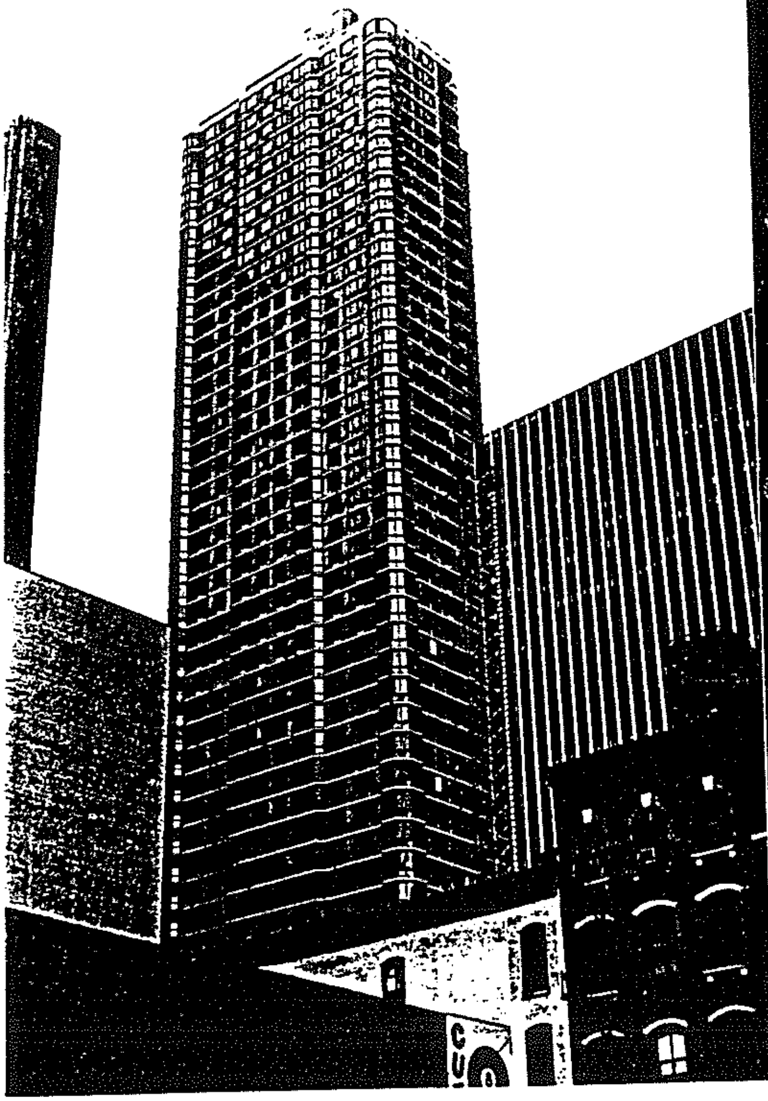
BRIEF BIOGRAPHICAL SKETCH

Jacob S. Grossman, FACI, PE, is partner in the firm of Rosenwasser/Grossman Consulting Engineers, P.C., New York, NY. The firm specializes in high-rise construction. He has been a member of this firm for 38 years.

Mr. Grossman is currently a member of the Reinforced Concrete Research Council; a past member of Concrete Material Research Council; a past member of ACI Committee 318, Standard Building Code and is currently a consulting member to this Committee. He is and has served as a member of several other Technical Committees at ACI and has also served on the ACI Board of Direction.

Mr. Grossman received the 1987 ACI Maurice P. van Buren Structural Engineering Award and in 1989 he received the Alfred E. Lindau Award.

Mr. Grossman holds degrees from the University of California at Los Angeles and the University of Southern California.

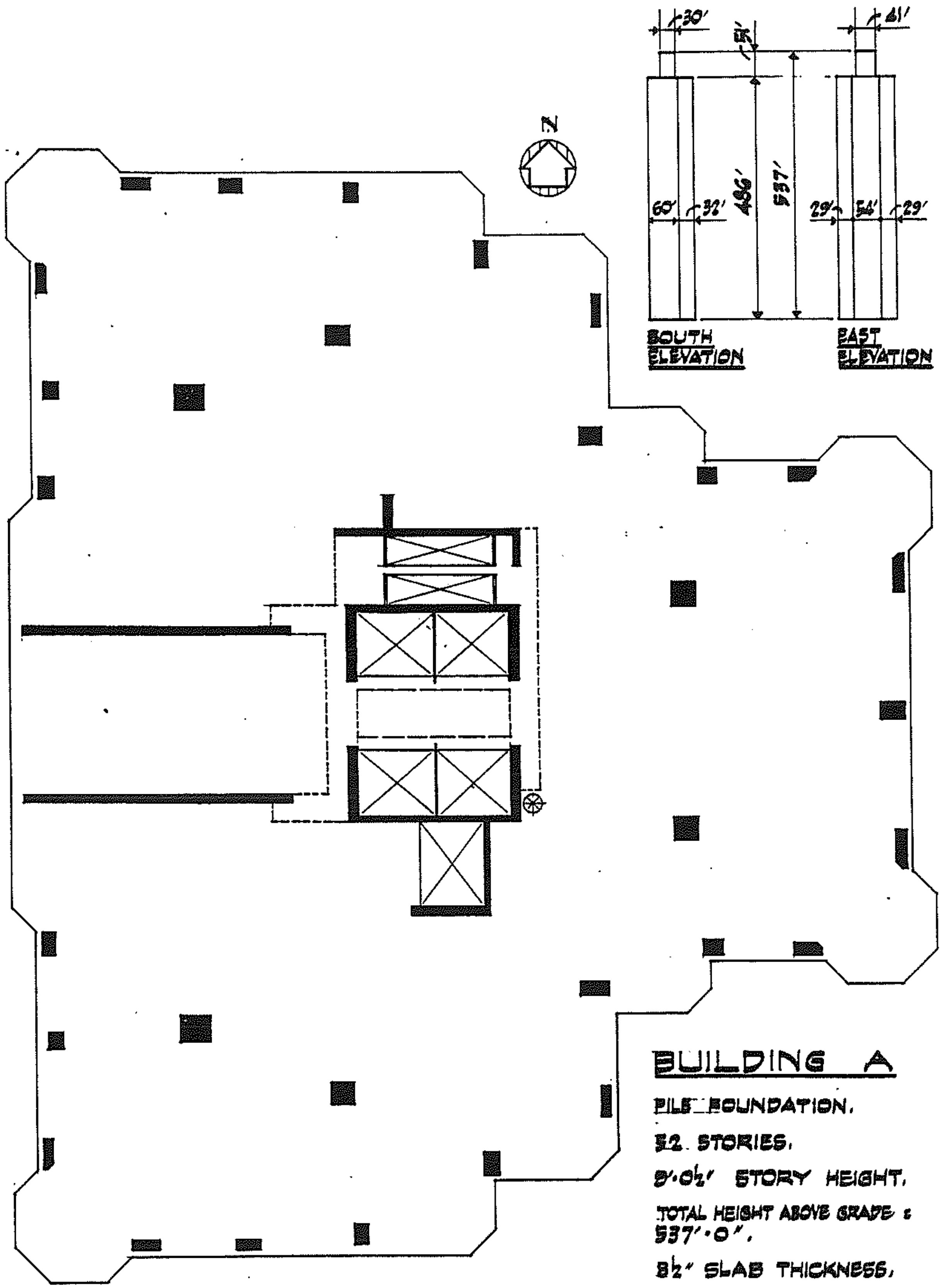


Bldg A

Fig B1

Bldg B

Fig B2



BUILDING A

PILE FOUNDATION.

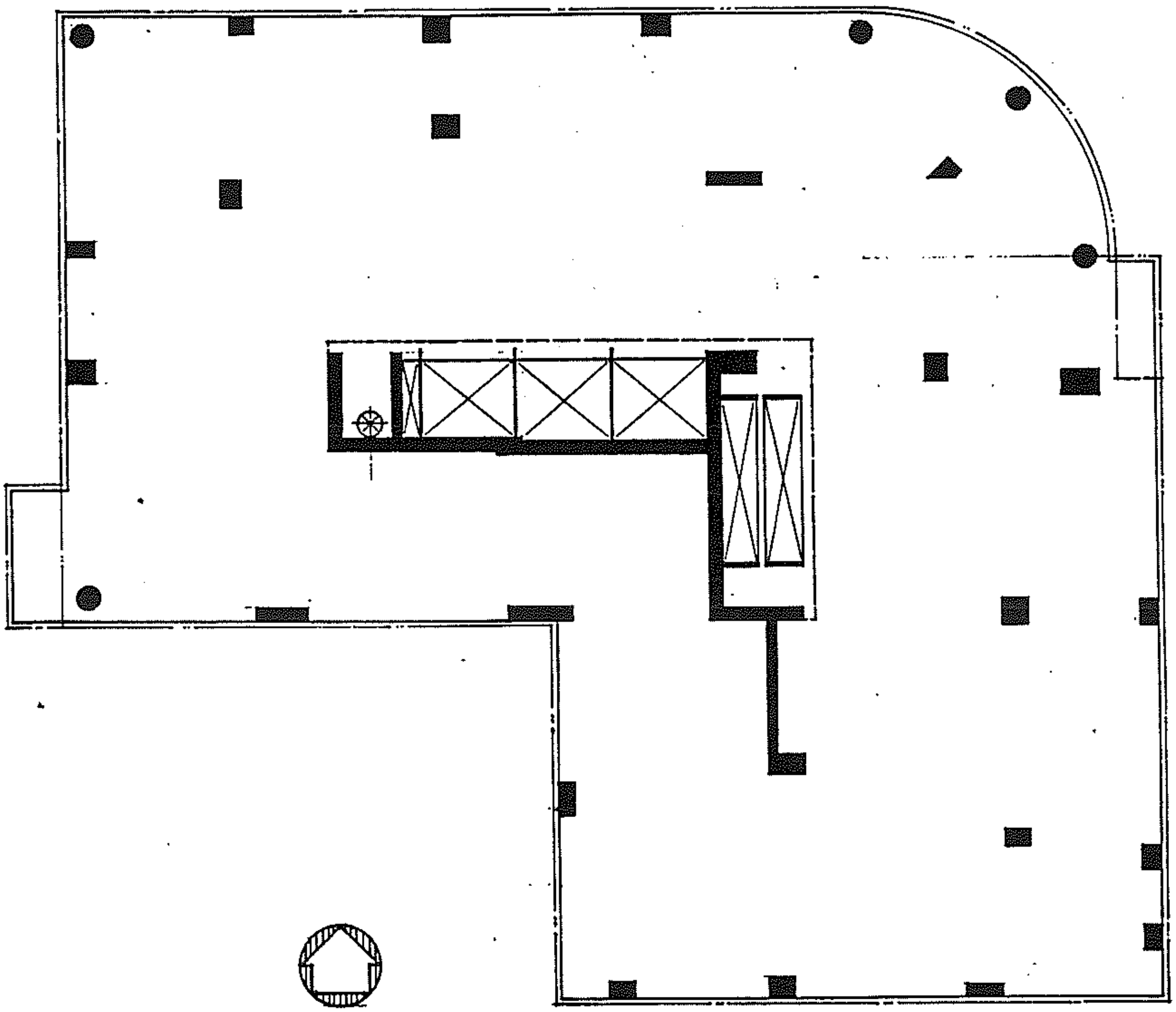
52. STORIES.

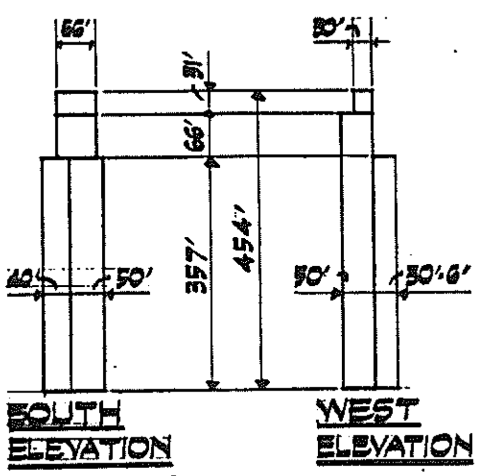
9'-0½' STORY HEIGHT.

TOTAL HEIGHT ABOVE GRADE: 537'-0".

8½" SLAB THICKNESS.

Fig E-3





BUILDING B

40 TON/SQ. FT. ROCK FOUNDATION,
40 STORIES,
9'-8" = STORY HEIGHT.
TOTAL HEIGHT ABOVE GRADE = 454'-0".
8½" = SLAB THICKNESS.

FIG E-7

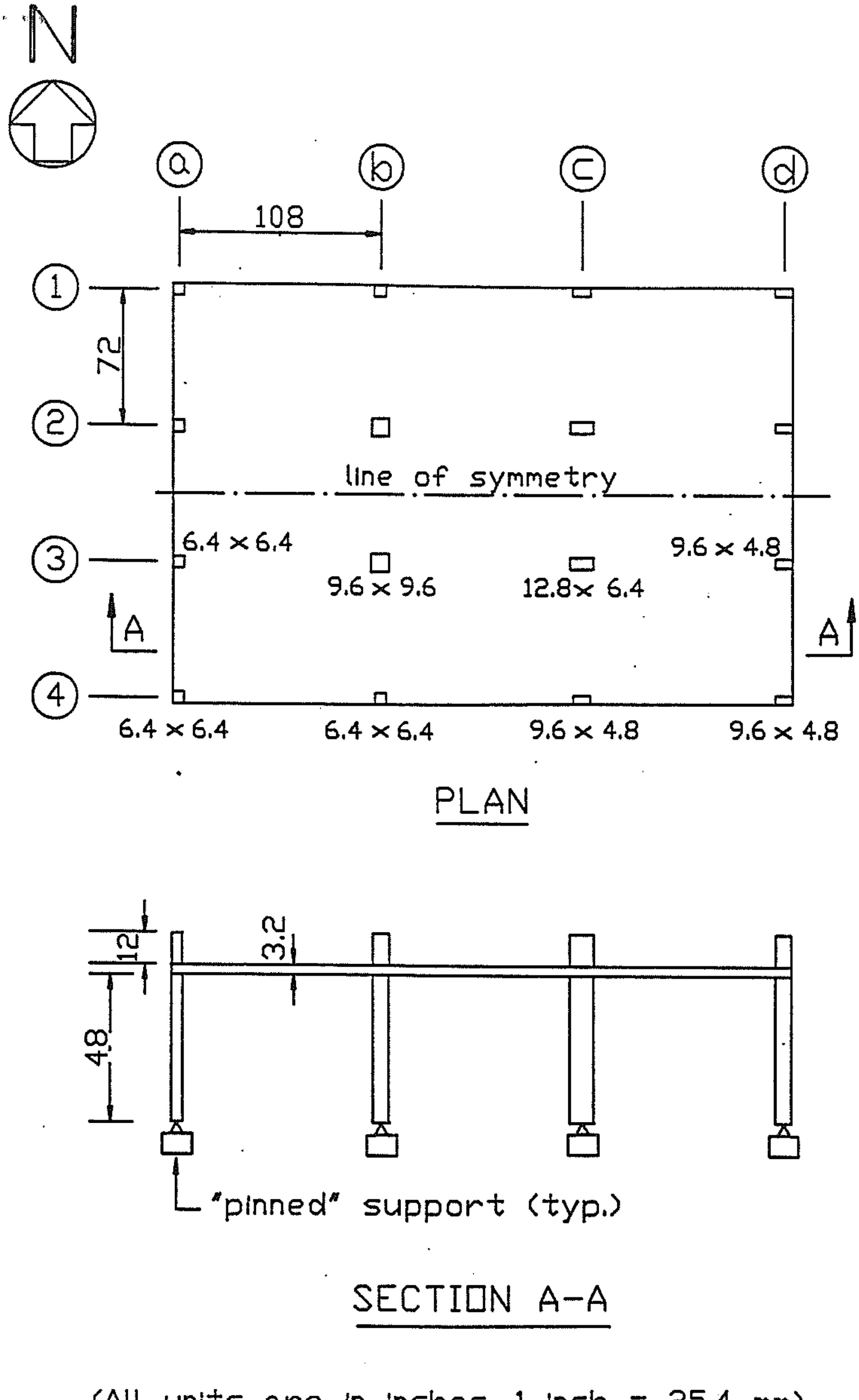


Fig. 1 Layout of Test Slab

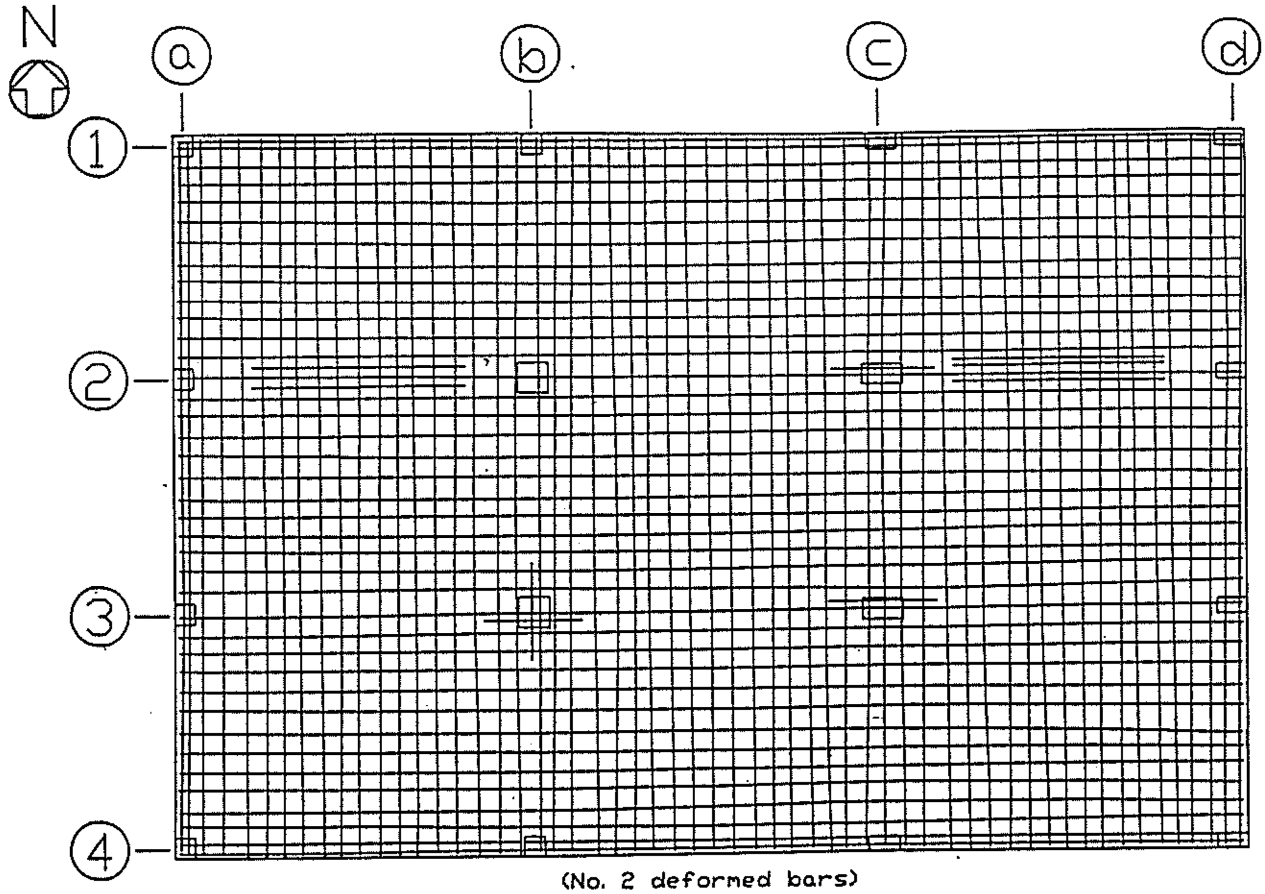


Fig. 2(b) Bottom Steel Mat of Test Slab

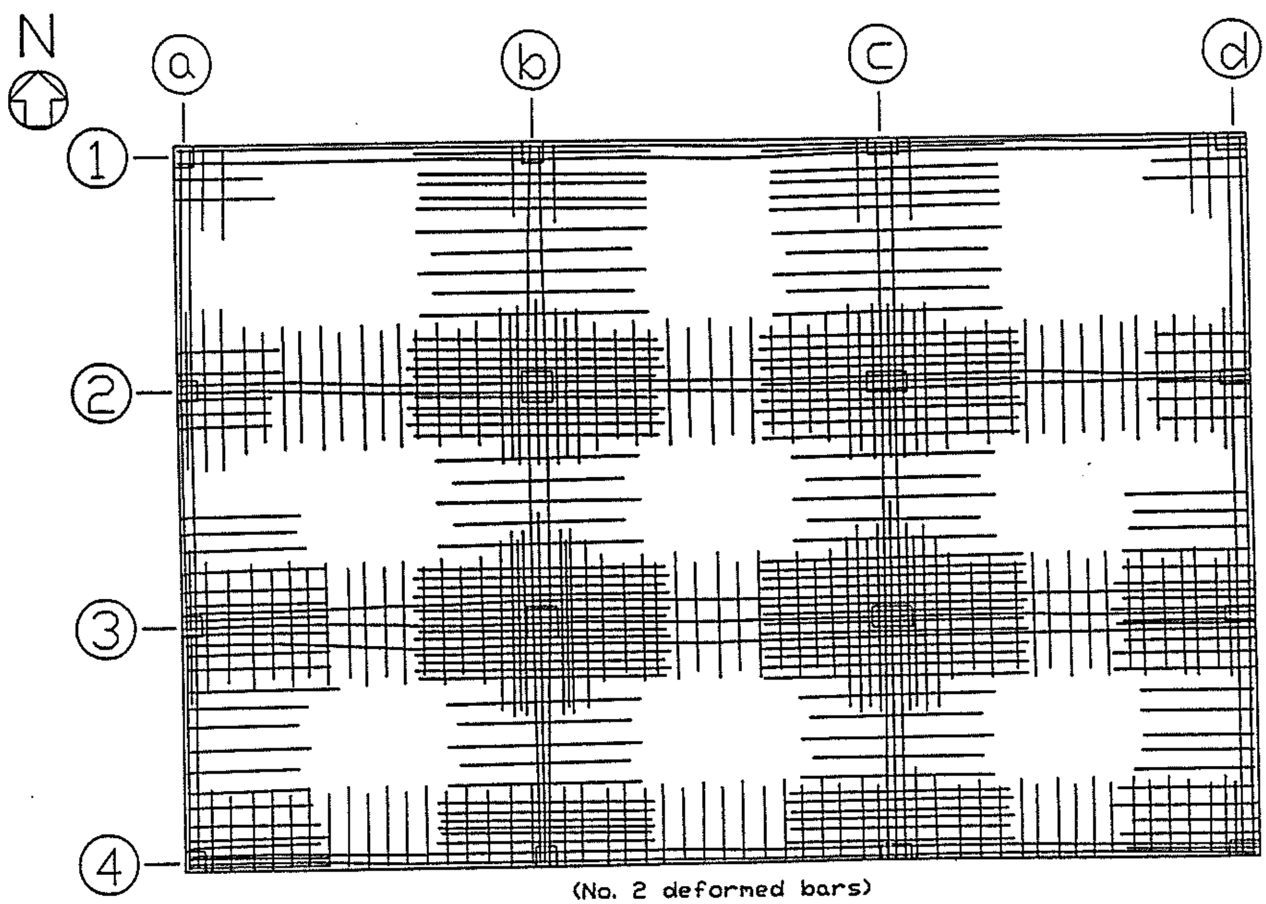
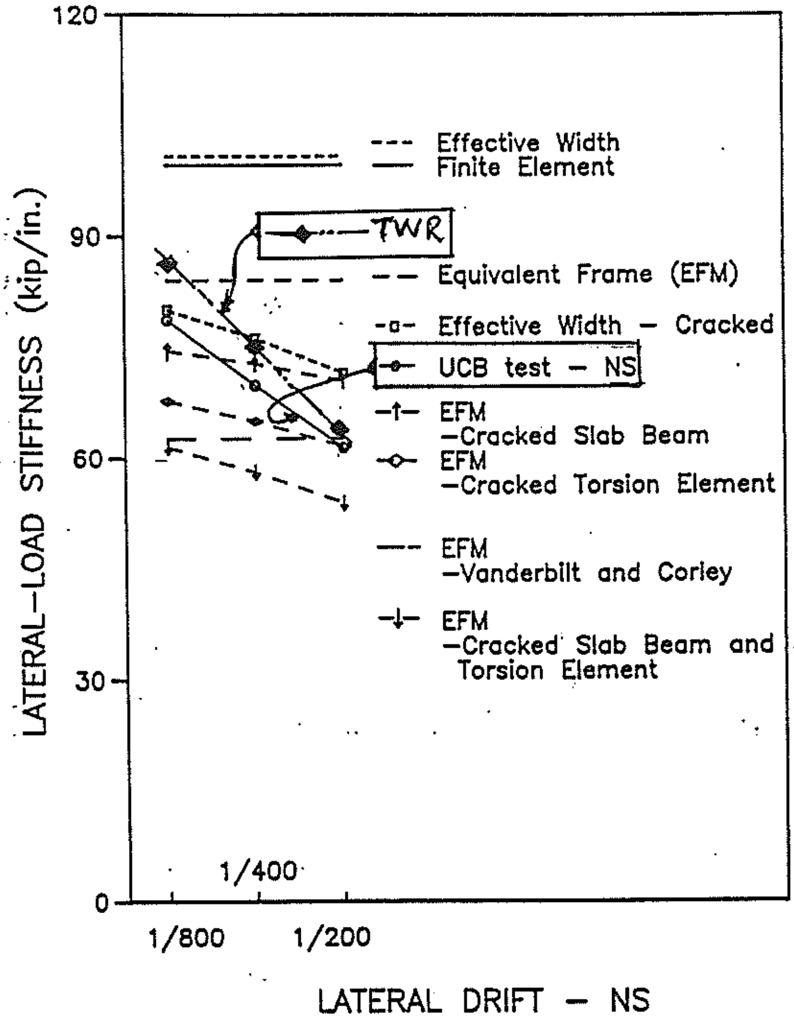
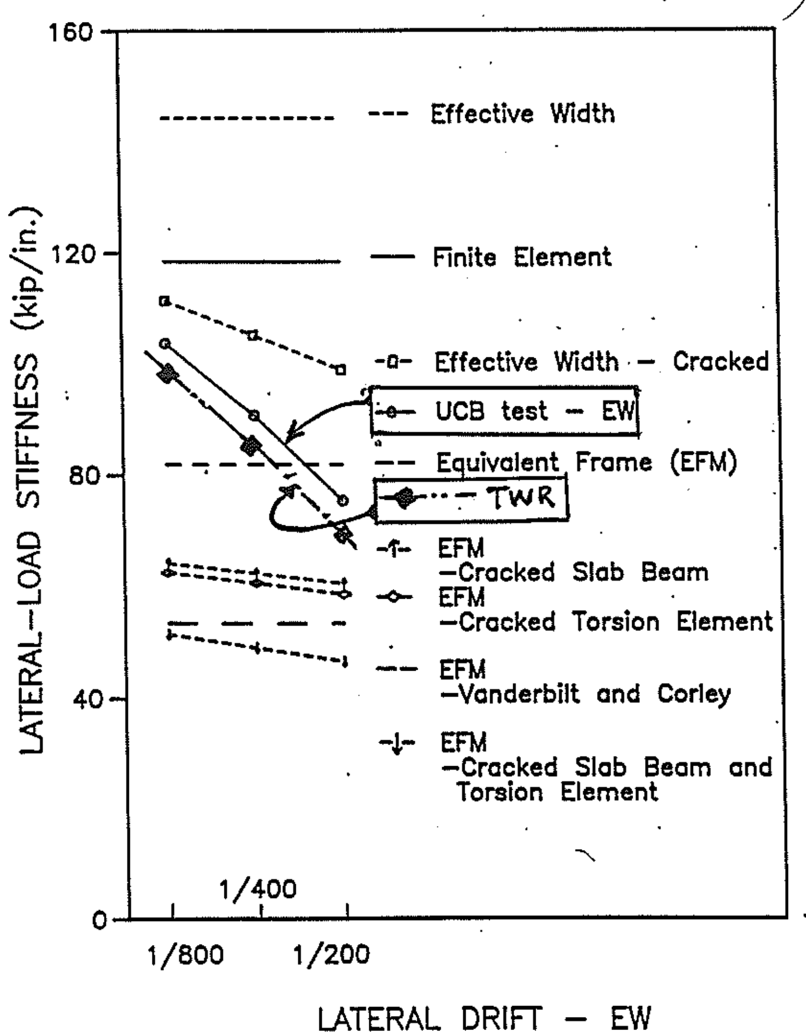


Fig. 2(a) Top Steel Mat of Test Slab



The Lateral Stiffnesses of UCB Test Slab (NS Dir.)

Fig 3



The Lateral Stiffnesses of UCB Test Slab (EW Dir.)

Fig 4

Fig. 1

Layout of Test Slab at UCB [1]

Fig. 2 (a)

Top Steel Mat of Test Slab at UCB [1]

Fig. 2 (b)

Bottom Steel Mat of Test Slab at UCB [1]

Fig. 3

Lateral Drift - NS Direction
comparing "TWR" with methodologies reviewed in UCB [1]

Fig. 4

Lateral Drift - EW Direction
comparing "TWR" with methodologies reviewed in UCB [1]

Fig. B-1

West Elevation of Building A

Fig. B-2

East Elevation of Building B

Fig. B-3

Lower Typical Floor of Building A

Fig. B-4

Lower Typical Floor of Building B

TABLE B1

MEASURED VS. COMPUTED PERIODS AT AMBIENT CONDITIONS

TABLE B2

MONITORING BUILDING A
During Wind Storm March 4, 1993

Table-1

Chronology Of Tests On The Flat Plate Tested At UCB [1]

Table-2

Measured Global Lateral Stiffness At Peaks [1]

Table - A

Stiffness Adjustments for North Half Slab-Column Connections

Table - B

Ratio of Cracked / Gross Column Moment of Inertia

$R_2 = I_{cr} / I_g$

Table - C

Stiffness Adjustments R_3** for Effective Panel Width in North Half of Test Slab

Table - D

Sensitivity Review - Comparing Shear in Columns. (h_s / 400)

Table - E

Sensitivity Review - Comparing Shear in Columns. (h_s / 200)

Table - F

Sensitivity Review - Comparing Shear in Columns. (h_s / 800)