

PAPER SP36-1

WHAT DO WE NEED TO KNOW ABOUT THE BEHAVIOR OF STRUCTURAL CONCRETE SHEAR WALL STRUCTURES

BY
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This paper, in essentially the present form, was presented as an introduction to a three-hour discussion session at the 1971 Conference on the Behavior of Concrete Structural Systems. The conference was conducted by the Department of Civil Engineering, West Virginia University and was jointly sponsored by the American Concrete Institute; American Society of Civil Engineers; Reinforced Concrete Research Council; Portland Cement Association; Prestressed Concrete Institute; and the National Science Foundation.

Synopsis: An attempt is made to point to some fundamental topics which are of particular concern to the designer of shear wall buildings.

There is little guidance available to the designer to evaluate the effect of transverse walls in a system of flanged shear walls.

In a system of coupled shear walls the relationship between the stiffness of the coupling elements and the rigidity of the total system is poorly understood. There are no available design tools for the case where the coupling is affected solely through slabs.

It is suggested that a better understanding of the shear transfer mechanism is needed for all types of slab/wall connections. For designs in seismic regions more knowledge is needed about post-cracking behavior and ductility of shear walls.

Keywords: beams (supports); connections; cracking (fracturing); ductility; earthquake resistant structures; flanges; flexural strength; lateral pressure; loads (forces); reinforced concrete; shear stress; shear walls; slabs; static loads; stiffness; strength; stress transfer; structural design.

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INTRODUCTION

Much is known about shear walls. We have investigated, designed and constructed some impressive shear wall structures. We are rightfully proud of these accomplishments. However, we also admit to serious reservations about some of our designs. While designing shear wall buildings, we find ourselves spending many hours worrying about our design.

The focal point of this presentation is on "what do we need to know." Of the many topics that plague the designer due to lack of knowledge, only a few specific items are presented here. This is not to say that other topics do not bother the designer--indeed there are many. The topics chosen for these remarks are, however, so basic that we must have a better understanding of them before proceeding to more exotic ones.

The design process of a shear wall structure has essentially four parts:

1. The conceptual design stage where the criteria are established and framing schemes identified. A tentative decision is made concerning the location and shape of shear walls.
2. The determination of forces assigned to each wall or system of walls: analysis of the selected structural scheme.
3. A review of the strength of each wall and wall component: check of unit stresses.
4. The detailing of the design.

The four parts of the iterative process are obviously interrelated. At any given time, the designer may think of one part only or all four at the same time. So it is with these remarks: the subject is segregated into three essential parts (omitting the conceptual design stage), but in any one part other parts are introduced.

ANALYSIS

It is common practice to express the seismic or wind forces in equivalent static forces. Thus, we quickly and conveniently arrive at the

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total lateral force acting on a structure. These total forces are then distributed into individual walls or systems of connected walls. If the walls are not linear or if they are not of a uniform height or cross-section, this seemingly simple process becomes a major problem.

Non-linear walls--Figure 1 gives a set of examples describing difficulties with non-linear walls.

Figure 1a shows four equal walls. If the floor diaphragms are rigid, each wall will obviously take 25% of the lateral load.

In figure 1b, the building has a closed wall system in the center. If the structure is 10 feet high, we will probably say that each of the walls parallel to the load takes 25% of the total load; the central core takes 50% of the total load. For a low building, the load distribution is the same as in the first case (figure 1a), even though the geometry of the walls is quite different.

If the building in figure 1b is 150 feet high, flexural deflections will dominate over shear deflections. Accordingly, we distribute the total load in proportion to the moments of inertia. The central core now takes 72% of the shear.

The obvious conclusion is that for this particular geometry, height does make a large difference. There is no hesitation in assigning to the central core 50% of the total shear in the 10-foot high building, nor in assigning 72% to the 150-foot high building. There would be some, but not serious, hesitation for a 60-foot high building.

In figure 1c, the central core is increased while the building remains 150 feet in height. Our first impulse here is to say that only a portion of the flange created by the longitudinal walls is effective. We must, however, be very careful. We may err on the unsafe side: ignoring the flange, we decrease the load attributable to the core. If the entire flange is ineffective, the central core takes 50% of the total load; if the entire flange is effective, the core takes 84% of the load. Ignoring the flanges would produce an underdesigned core and overdesigned end walls.

In figure 1d, the central core is in a C-shape. Certainly, the entire 70-foot flange is not effective! Or is it? The width of the effective flange is subject to individual interpretation. Perhaps the best answer is to average many individual judgments. If so, no one can be more than half wrong! But, the design may be only half right! (At the 1971 West Virginia University Conference on the Behavior of Concrete Structural Systems, the 50 participants were asked to volunteer an impromptu opinion concerning the effective length of the flange. The answers ranged from 10 feet to 70 feet. Most answers were in the 30 to 50 feet range. Only a

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few participants could present an impromptu opinion when the building height was reduced to 70 feet.)

Coupled walls--Coupled shear walls are very common in residential buildings. A simple coupled shear wall is shown in figure 2. We must know the rigidity of the system to design the wall. The rigidity of the system is computed for various connecting beam stiffnesses (1). The results are tabulated in figure 2 and are plotted in figure 3. Note that the rigidity of the system is not sensitive to the stiffness of the connecting beams as long as these beams have "appreciable" stiffness. It is hard to define "appreciable." If the connecting beam is a flat slab, "appreciable" may be an element whose width is 15 thicknesses.

While the curve in figure 3 is drawn for the specific example described in figure 2, it is quite typical. The relation of beam stiffness to the system stiffness changes rapidly in a rather vague region where the connecting beams change from "stiff" to "slender." There is a lot of grey between "white" and "black"; and there is much latitude between "stiff" and "slender." It is thus important to the designer to accurately determine the stiffness of the connecting beam. This becomes particularly important when slabs only couple the walls, since a small difference in assumed effective slab width will greatly influence the rigidity of the entire system.

Barnard and Schwaighofer have tested a 1/64 model of a coupled wall (2). The modeled structure of their test is shown in figure 4. The width of the effective beam was varied from 6.5 slab thicknesses to 32 slab thicknesses. They found that the entire slab width of 32t is effective in coupling the walls. Somewhat related to the same question are studies and tests by Kahn and Sbarounis (3). Figure 5 shows particulars. Kahn and Sbarounis suggest an effective width of slab for a slab/column system (as opposed to a slab/wall system). An extrapolation of Kahn's and Sbarounis' data indicates that as the length of the column (or perhaps wall) increases, a greater portion of the slab becomes effective in uniting the columns into a frame (or perhaps coupling the walls). While this extrapolation may be unwarranted and shaded with an artistic license, it nevertheless supports Barnard's and Schwaighofer's conclusions.

However, Barnard and Schwaighofer point out that their conclusions are valid only for a relatively narrow door opening (2)(4). Kahn and Sbarounis point out that the span (perhaps width of corridor) is an important factor. The designer thus appreciates that many parameters have an influence on the effective width of the coupling beam. To date, however, he has no specific guidance to estimate the effective width of the connecting slabs in situations other than those tested. The practical detailing problems of the extremely wide coupling unit are discussed in the next section.

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STRENGTH EVALUATION

Having determined forces in each wall, first-trial sizes are tested against allowable unit stresses.

Historically, for 4000 psi concrete, maximum allowable unit stresses in shear walls have been:

1958 Uniform Building Code:

$$v = .05 f'_c = 200 \text{ psi}; v_u = \text{approximately } 400 \text{ psi}$$

1967 Uniform Building Code:

$$v_u = 5.4 \phi \text{ to } 10 \phi \sqrt{f'_c} = 290 \text{ to } 540 \text{ psi}$$

1971 American Concrete Institute Building Code:

$$v_u = 10 \sqrt{f'_c} = 630 \text{ psi}$$

In 1958 the basis for design was intuition.

In 1967 shear walls were compared to deep beams and smaller unit stresses were allowed in short walls than in tall walls.

In 1971 the effects of axial loads and flexure were introduced.

Keeping in mind the height of the wall (principal tensile stress for short walls; flexural tensile stress for high walls), it can be said that the order of magnitude of allowable unit stresses has not really changed. From a practical point of view, the designer is not overly concerned with unit shear stresses in walls. If concrete stresses are high, he uses more steel. If the total shear stress, v_u , is too high, a thicker wall is designed.

The code unit shear stresses for walls are quite well accepted. Shear walls have not failed due to high unit shear stresses. Other more important factors have caused the failures.

It is while examining unit stresses, however, that we particularly realize that a wall is only part of the shear-resisting system. Shear forces are introduced into walls through slabs; often a floor may transfer large shears from one wall to another. These slabs are called "diaphragms"; diaphragms are really shear walls which happen to be oriented horizontally.

It is not at all unusual to have a wall loaded with one unit of shear at a particular floor and loaded with two or three units at the next floor below. The additional shear is introduced by a slab. The designer ponders: what should be the allowable in-plane unit shear stress in a slab in a

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region of high negative moments and high shears due to vertical load? He is not quite sure of the mechanism which transfers forces from the slab to the wall. Is it through the reinforcement? Through that portion of the concrete which is not cracked? Both? Neither?

Often the reinforcement in the slab is sized for negative moment; no additional reinforcement is provided for in-plane shears. Is this practice correct?

Assume that the in-plane shear capacity of a 4-1/2-inch slab is known. Will then a 30-inch waffle system with a 4-1/2-inch slab have a higher capacity? How much higher? What about a pan-joint system? Do the joists contribute to the shear strength of the slab?

Concerning unit stresses, designers are very confused in the design of the connecting beams occurring between coupled walls. Invariably, vertical shears in these beams are very large. Thomas Paulay of New Zealand has tested some 3/4 scale models of rectangular beams (5). The beams had conventional reinforcement: top and bottom horizontal steel with closely spaced closed stirrups. Paulay reports: "The behaviour of such beams in many respects defies the customary concepts of reinforced concrete design." This really shakes the confidence of the designer!

If the horizontal load acting on the coupled shear wall shown in figure 2 is 0.5 kips per vertical foot of wall, the ultimate shear stress in the wall with a load factor of 2 is 39 psi. This is a low stress level. The designer is very happy, becoming convinced that there are no problems. He proceeds to review the beams connecting the two walls. With a load factor of 2:

For a beam 12 inches wide x 24 inches deep, $V_u = 60$ kips.

For a beam 10 feet wide x 8 inches deep, $V_u = 54$ kips.

The problem is stated in figure 6. In the first case, keeping in mind Paulay's report, we will produce a conservative but reasonable design.*

In the second case, our sixth sense tells us that the solution is not quite so clear. We have a feeling from Barnard's and Schwaighofer's work that the 10-foot wide slab (15t) is effective in connecting the walls, but we lack the tools to confirm our feeling. We start to think of torsion,

*Subsequent to the presentation of this paper, the author, through private correspondence from Dr. Paulay, became aware of Dr. Paulay's further work wherein he suggests design procedures for rectangular beams of the same width as the coupled walls (6).

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warping, distribution width for flexure, and many other interesting problems. At best, our solution is devious.

DETAILS

At this point, the designer departs from the "science" of engineering and enters the "art" of engineering. "Art" is here defined as a mixture of intuition, technology, empirical knowledge, experience and a flair for sculpture.

A series of simple questions illustrates the numerous problems:

1. For a wall heavily loaded in shear with little axial load and overturning moment, how much vertical reinforcement should be used for a given amount of horizontal reinforcement?
2. What is the optimum distribution of vertical reinforcement? More steel at the ends, less at the center? Uniform distribution?
3. A very basic question: In a 10-inch wall, should one layer or two layers of reinforcement be used? Consider also the practicality of placing two layers of reinforcement.
4. What type of construction joint should be used?

These are very basic detailing questions constantly facing the designer. Yet, we cannot answer them.

SPECIAL PROBLEMS IN EARTHQUAKE ZONES

Engineers designing for seismic loads face special problems. Since the imposed seismic loads may well be several times greater than the elastic strength of shear walls, consideration must be given to post-cracking behavior (strength, stiffness, load reversal, etc.) of the shear wall and detailing must provide ductility. Very little is known about wall ductility and post-cracking behavior.

An interesting sport among engineers is to attack the code. Usually any code will do. West Coast engineers usually choose to attack the Uniform Building Code. There are two favored targets: (1) the provision which limits shear wall buildings to 160 feet in height; and (2) the provision which doubles the overload factors for shear.

The sport is very popular. In more reflective moments, however, we sometimes admit that we are happy that these provisions legally limit our ambitions.

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CONCLUSION

In discussing our lack of knowledge concerning shear walls, only a few topics have been presented. The selected topics are quite fundamental. The designer faces them constantly.

Summarizing these problem areas:

1. For flanged shear walls, guidance is needed to assess the true effect of the flange. Some criteria similar to the American Concrete Institute "Requirements for T-beams" are needed.
2. In shear-walls coupled only by a slab, a better understanding of the relationship of the connecting beam to the wall system needs to be developed. New tools to design these connecting slabs are also needed.
3. In slab-to-wall connections, the shear transfer mechanism needs further explanation.
4. Finally, further laboratory testing is needed to find the best proportions of the familiar ingredients we use in our matrix: reinforcing steel and concrete.

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REFERENCES

1. Coull, A. and Choudbury, J. R. "Stresses and Deflections in Coupled Shear Walls," ACI Journal, Proceedings, Vol. 64, No. 2, February 1967, pp. 65 - 72.
2. Barnard, P. R. and Schwaighofer, J. "The Interaction of Shear Walls Connected Solely Through Slabs," Proceedings, Symposium on Tall Buildings (University of Southampton, April 1966), Pergamon Press, Oxford, England, 1967, pp. 157 - 180.
3. Khan, F. R. and Sbarounis, J. A. "Interaction of Shear Walls and Frames," Journal of the Structural Division, ASCE, Vol. 90, No. ST3, Proc. Paper 3957, June 1964, pp. 285 - 335.
4. Schwaighofer, J. "Shear Wall Structures," Structural Concrete Symposium, University of Toronto, Ontario, 1971, pp. 119 - 145.
5. Paulay, T. "The Coupling of Reinforced Concrete Shear Walls," Proceedings of the Fourth World Conference on Earthquake Engineering, Vol. 1, B-2, pp. 75 - 90, Santiago, Chile, January 13 - 18, 1969.
6. Paulay, T. "An Approach to the Design of Coupled Shear Walls." A paper received in private correspondence from Paulay.

METRIC EQUIVALENTS

Multiply	By	To Obtain
inch	2.54	cm
foot	0.3048	m
kip	453.6	kg
psi	0.07031	kg/cm ²
foot ⁴	0.008631	m ⁴

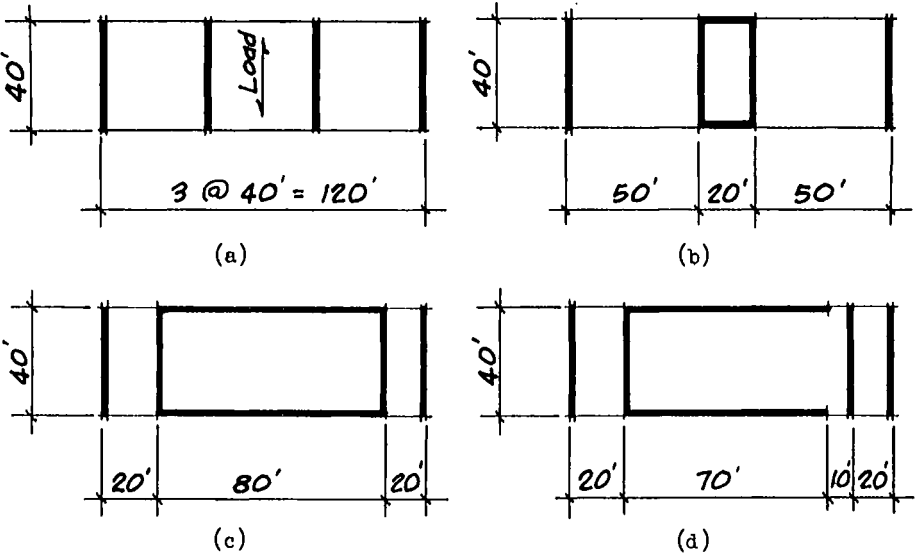
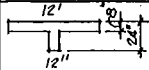
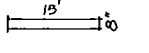
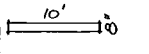


Fig. 1. Non-linear shear walls-rigid concrete diaphragm in all cases-walls same height and same thickness

WALL	CONNECTING BEAM	Rigidity of System
1	Solid 40' Wall No opening	100
2	 $I = 1.63 FT^4$	92
3	 $I = 0.67 FT^4$	92
4	 $I = 0.37 FT^4$	91
5	 $I = 0.25 FT^4$	87
6	 $I = 0.025 FT^4$	50
7	2 Walls @ 18' No Beams $I = 0$	18

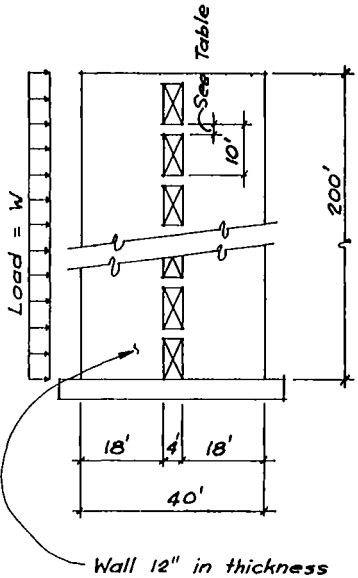


Fig. 2. Coupled walls-connecting beam stiffness vs. rigidity of system

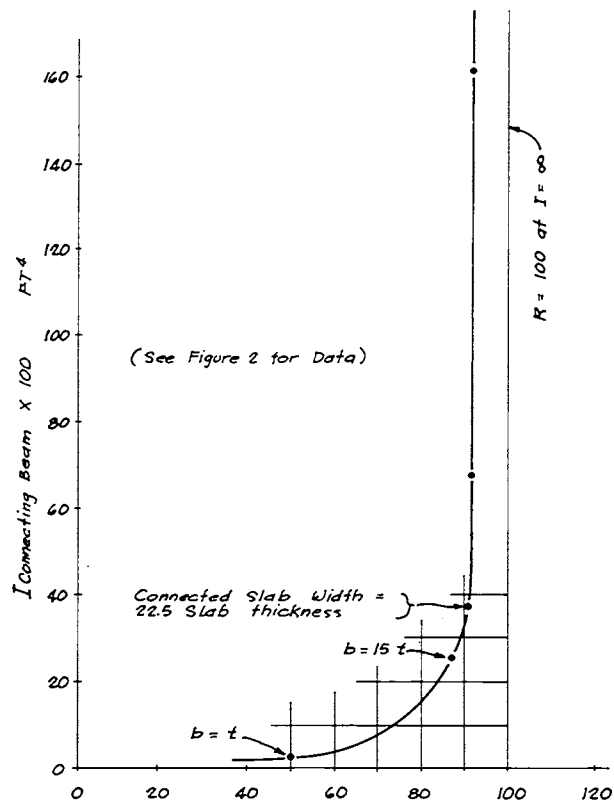


Fig. 3. Rigidity of wall system vs. connecting beam stiffness

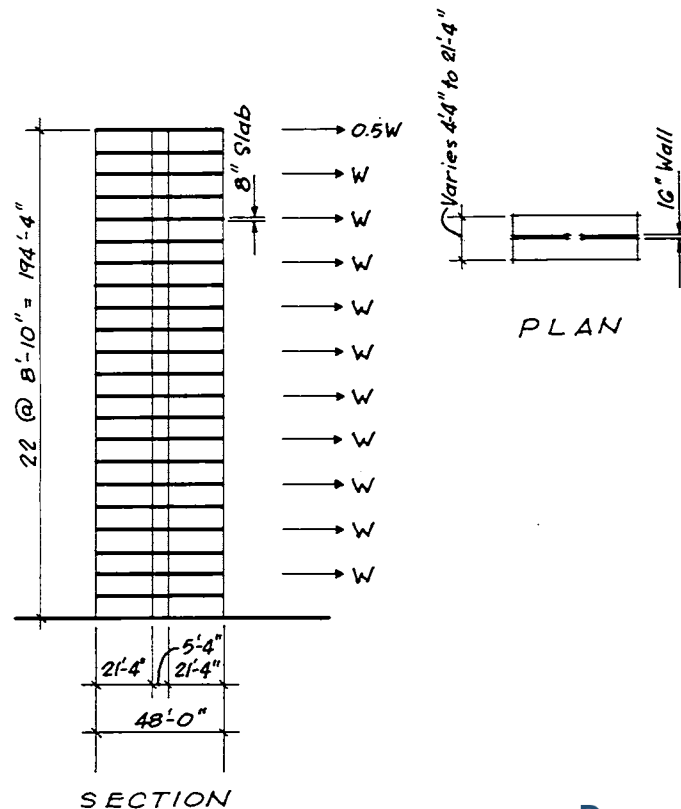


Fig. 4. Walls investigated by Bernard & Schweighoffer
-model=1/64 structure

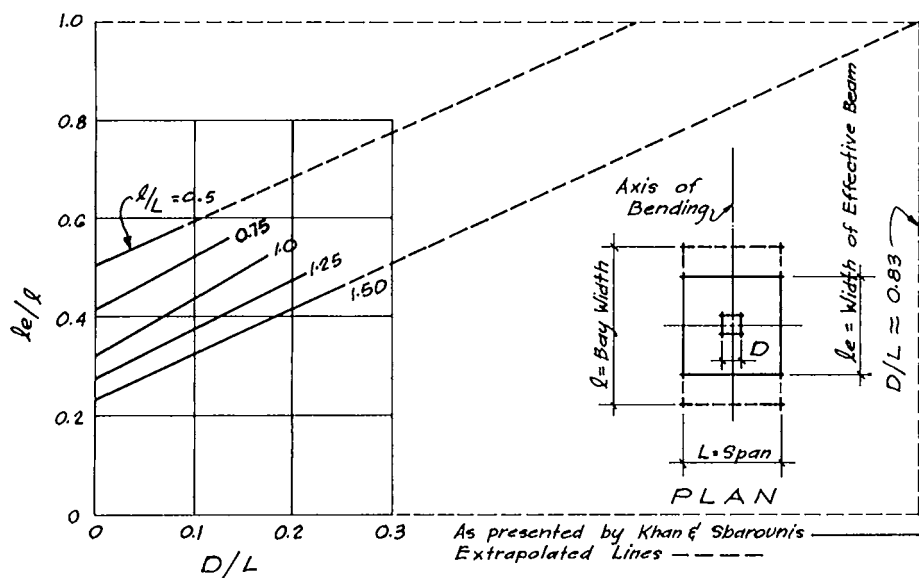


Fig. 5. Frame action in slab-column system (Khan and Sbarounis)

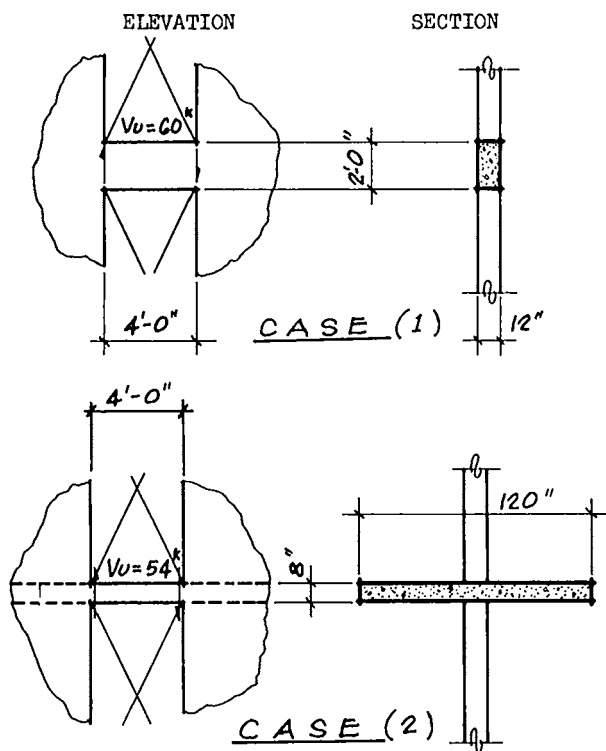


Fig. 6. Design of connecting beams in coupled shear walls

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FRAMES AND FRAME-SHEAR WALL SYSTEMS

BY
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Synopsis: This paper briefly reviews some aspects involved in the planning and analysis of frames and frame-shearwall systems, with particular reference to the lateral-load-resisting function. The basic structural systems commonly used in high-rise buildings are examined and a discussion of frame-shearwall behavior as well as some simplifying assumptions used in their analysis is presented. Areas requiring further study are indicated.

Keywords: beams (supports); columns (supports); concrete slabs; deflection; ductility; earthquake resistant structures; frames; high rise buildings; infilled frames; loads (forces); multistory buildings; reinforced concrete; shear properties; shear walls; stiffness; structural analysis; structural design; torsion; tube-in-tube buildings; wind pressure.

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FRAMES AND FRAME-SHEARWALL SYSTEMS

The purpose of this paper is to briefly review in fairly general terms some aspects involved in the planning and analysis of frame-shearwall systems, particularly as these relate to their lateral-load-resisting function, as well as indicate areas where further studies are required.

The term "frame-shearwall structure" will be used to refer to any combination of frames and shearwalls. In this context, the unbraced frame may be considered a special case of a frame-shearwall system, in which the shearwalls have about the same stiffness as the regular columns. Shear-wall buildings or cross-wall structures, consisting of a series of parallel walls linked transversely by floor slabs represent the other extreme in which all the vertical elements have considerable stiffness in the plane of the shearwalls.

While frame structures, depending mainly on the rigidity of member connections for their resistance to lateral forces, have been built to a height of 60 stories (1), they generally tend to be uneconomical beyond 10 or 15 stories. This is particularly true of the conventional form of frame structures consisting of two sets of mutually perpendicular frames. This limitation has become more apparent in recent years with the practice of using lightweight elements or glass for exterior walls and interior partitions. Since the ideal multistory structural system is that which pays the least premium for height over that required to support the gravity loads, in order to provide the necessary stiffness against lateral loads, recourse usually has to be made to other configurations for buildings exceeding, say, 10 stories in height.

The introduction of deep vertical elements or shearwalls represents a structurally efficient solution to the problem of stiffening a frame system. However, except perhaps where the shearwalls are located along the ex-

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terior of a building, some degree of planning flexibility may have to be sacrificed with their use. In many cases, a judicious disposition of shearwalls in plan allows these to function efficiently as vertical and lateral load resisting elements without interfering much with architectural requirements. Figure 1 shows typical plan arrangements of frame-shearwall systems (2). Reinforced concrete structures using systems similar to these have been built to a height of 70 stories. Many frame-shearwall structures of lesser height have performed very well under strong earthquake loading.

A modification of the conventional frame arrangement which has been found economically suitable for buildings up to about 50 stories (approx. 500 ft.) high is the so-called "framed tube" (3, 4). In this structure, a typical plan of which is shown in Fig. 2, the exterior columns in what would otherwise be a conventional frame structure are spaced more closely together and are connected by relatively deep spandrel beams to form an exterior grid which is usually designed to resist the entire lateral load. The framed tube represents a logical evolution of the conventional frame structure, possessing the necessary lateral stiffness with excellent torsional qualities while retaining the planning flexibility which isolated interior columns allow. Because the major component of resistance to lateral deflection derives from the flexural action of relatively shallow members, the framed tube is somewhat inefficient from a purely structural point of view. However, because the exterior grid of closely spaced elements usually also serves as mullions for the exterior glazing in most buildings, significant savings are effected which tend to offset any inefficiency in the structural system. The use of inclined elements in the exterior grids allows these to resist a greater percentage of the lateral load by axial forces rather than bending and makes for a more efficient structure.

For taller structures, an arrangement which has evolved during the last two decades or so and which has been found particularly suitable for office buildings, is the so-called hull-core or framed-tube-with-core structure. A typical plan is shown in Fig. 3. This system has emerged as a logical solution to the problem of providing a tall, stiff structure with wide column-free spaces between a central core which houses all services and an external peripheral grid of closely spaced elements which can serve as mullions for the facade glazing.

For still taller structures, especially where a large plan area is involved, intersecting planes of interior shearwalls or closely spaced column-beam grids traversing the entire width of the building may be used. (Fig. 4) Connecting the interior shearwalls or grids with the exterior pe-

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ripheral grids reduces the shear lag* across the windward and leeward grids (see Fig. 5) and allows the latter to participate to a greater extent in resisting the lateral load. The use of such interior vertical diaphragms when indicated, essentially produces a vertical, multiple-cell cantilever box beam.

PLANNING CONSIDERATIONS

Although architectural requirements usually play a dominant role in the disposition of major structural elements in a plan, substantial savings can often be achieved through an efficient arrangement of the structural elements while still satisfying architectural needs. As mentioned earlier, from the structural-economic point of view, that structural system should be aimed for which pays the least premium for height. By premium (see Fig. 6) we refer to the increase in structural member sizes, over that needed to support the gravity loads, usually required in tall buildings by lateral loads. These requirements have to do mainly with providing sufficient lateral stiffness in order to limit the "drift" or lateral deflection to an acceptable value. Also involved is the need to keep member stresses due to the combined action of lateral and gravity loads within the permissible increased values specified in codes.

Drift limitation -- Because it is the principal criterion used in assessing the proper lateral stiffness to be built into tall buildings under wind loads and may determine the type of structural system to be employed, the amount of lateral deflection or drift which is deemed acceptable deserves more than passing consideration. The use of a maximum allowable drift is based on the need to limit to safe or tolerable levels the effects of lateral sway on (a) the stability of individual columns as well as the structure as a whole, (b) the integrity of "non-structural" partitions and glazing, and (c) the comfort of the occupants of such buildings. No systematic study has

* The phenomenon which, in frames subjected to lateral loads, exhibits itself in the relatively rapid decrease in the vertical forces transmitted to the columns as one moves toward the center of the frame. This results from the inherent flexibility of the frame elements, the flexural deformations in which cause a significant reduction in the shear transmitted across the frame, when compared to that which would be calculated on the basis of an assumed cantilever beam behavior.

yet been made to determine the precise relationship between drift and the above three factors, although some work has been done on the lateral stability of frames (5) as well as on human sensitivity to motion (6). To date, only the National Building Code of Canada 1970, among North American model building codes, specifies a maximum value of the deflection index corresponding to the design wind loading (in this case, $1/500$). The report of ACI Committee 435 on "Allowable Deflections" (7) recommends a maximum of $1/500$ for the deflection index.

In the absence of code-prescribed limitations on drift due to wind, or provisions delineating the method of calculation to be used or the assumptions to be made in determining lateral deflection, deflection indices ranging from about $1/300$ to $1/600$ (8) have been used in practice, depending upon the judgment of the engineer. The higher value of about $1/300$ appears to be more appropriate for the traditional building types of several decades ago where heavy so-called "non-structural" masonry infilling substantially increased the lateral stiffness of frames. The recent trend of using lightweight elements and glass for interior partitions as well as exterior cladding, coupled with the more precise determination of deflection made possible by the electronic computer would suggest the use of a relatively lower limit.

An evaluation of the relationship between drift and the pertinent design parameters based on existing buildings would have to consider the design wind force used, the method of calculation and the assumptions made in computing lateral deflections as well as the actual wind conditions under which such structures perform. The increasing availability of computer programs for lateral load analysis (9, 10, 11, 12) should help eliminate the differences in the manner of computing deflections and should lead to a more precise definition of an allowable drift or deflection index.

Basically the same considerations as mentioned for wind enter in a design for earthquake loads, although one might expect slightly more liberal drift requirements under major earthquakes. For severe earthquake motion, the principal consideration insofar as drift is concerned is the stability of the structure under the action of gravity loads when undergoing large lateral displacements. Appendix D of the SEAOC Code (13) mentions an allowable drift due to the specified earthquake forces twice that allowed for wind. In applying such a limiting value, a distinction should be made between the drift produced by the code-specified static forces and the dynamic lateral displacements corresponding to a particular earthquake. The latter could be several times larger than the former (14).

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Avoiding unnecessary torsion -- Because torsion due to lateral loads can produce significant stresses, particularly in corner columns, a more or less symmetrical arrangement of the principal lateral load resisting elements with respect to the entire plan should be aimed for. In this connection, the torsional resistance of a structure would be much improved by locating the lateral load resisting elements where they do the most good, i. e., along the periphery of a building. This, of course, assumes that the floor slabs are so constructed that they provide an effective means of transmitting the horizontal forces to the major resisting elements and make these act as a unit. The effect of torsion is particularly important in buildings located in seismic regions.

Loading the shearwalls vertically -- Shearwalls are efficiently utilized if they are distributed about the plan so that they carry their proportional share of the vertical load rather than having them function mainly as lateral load resisting elements. This would also tend to reduce the differential shortening due to creep between a shearwall and adjacent columns and hence the associated forces in horizontal connecting members which are rigidly attached to these vertical elements. The use of a centrally located vertical shaft or corewall to enclose the services at each floor allows the shearwall to carry a major portion of the gravity loads while also serving to stiffen the building against lateral loads. In buildings which are rectangular or oblong in plan, two such corewalls located near each end have been used to advantage.

Tying frame and shearwall -- Where a shearwall or corewall is centrally located and columns occur along the plan periphery, a significant increase in lateral stiffness can be achieved by tying the shearwall and the outlying columns with deep flexural members at the top and perhaps at intermediate levels for very tall buildings. This linkage, which is illustrated in Fig. 7 more effectively utilizes the longitudinal (axial) stiffness of the peripheral columns in resisting the lateral loads (1). Deep flexural elements serving as shear-webs between vertical elements have also been used in tall frame structures. The main object here is to cause the structure to act more as a vertical cantilever beam and so resist a larger proportion of the lateral loads by axial forces rather than bending in the columns. Clearly, the former mode of resistance represents the more efficient use of materials.

In this connection, it is interesting to note the significant contribution which peripheral columns can provide to the lateral resistance of a tall building. Figure 8 shows the calculated deflections based on the model

test (15) of the 46-story, 590-ft. "Australia Square" tower in Sydney. The curve marked "A" indicates the lateral deflection under a 1.0 k/ft. horizontal load on the model of the central circular core, before the outer ring of columns was attached. Curve "B" shows the deflection under the same load on the completed model, after the peripheral columns were connected to the core through the beam-slab system. At the top story, the stiffening effect of the connected exterior ring of columns produces a reduction of some 65 percent in the lateral deflection.

TYPICAL BEHAVIOR OF ELEMENTS

An understanding of the behavior of frame-shearwall structures under lateral load can be very helpful when performing an approximate analysis or when interpreting the results of a computer analysis. Figure 9 shows the typical deformation patterns exhibited by frames and shearwalls when subjected separately to lateral loads and the horizontal interaction forces which are developed when both elements are connected by floor slabs which enforce equal lateral deformation at the floor levels. The interaction between the two elements is such that the frame tends to reduce the lateral deflection of the shearwall at the top while the wall supports the frame near the base.

A typical variation of the shears carried by each element in a frame-shearwall system under lateral load is shown in Fig. 10. It is significant to note that the total shear carried by the frame at the top stories can exceed the applied story shear at these levels. Clearly, distributing the applied shear to the different resisting elements in proportion to their relative stiffnesses in this case can lead to grossly erroneous results. On the other hand, assuming the shearwall to carry all the lateral load can result in an underdesigned frame.

Figure 10 also shows the sizable increase in the horizontal shears which occurs in the lower stories of a system where a shearwall is supported at the base by columns. The shears shown for the lowest story of the shearwall are those corresponding to the supporting columns. The character of the shear distribution curves for the lower stories is typical of the force concentrations which occur in regions of major geometric or stiffness discontinuities in a structural system. While yielding of the columns will tend to relieve the high stresses accompanying such large shears, the fact that these forces can exceed the total base shear for such configurations is worth noting. In structures subjected to earthquakes, such an arrangement can lead to early yielding in the columns, with corresponding large ductility

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requirements.

The nature of the interaction between frame and shearwall in a frame-shearwall structure is such that increasing the shearwall stiffness relative to the frame does not produce a proportional increase in the overall lateral stiffness of the structure. The plot shown in Fig. 11, which was taken from a study by Clark and MacGregor (16), shows the variation of the calculated deflection at the top of a 20-story frame-shearwall structure with the applied static load, here expressed as the ratio of the applied load to the assumed service load. The curve marked "H50" represents the behavior of a structure in which the ratio of the shearwall stiffness to the sum of the column stiffnesses in the frame at each story equals 50. The curves marked "H1" and "H2" correspond to structures in which the shearwall-to-column stiffness ratios are 1 and 2, respectively. This figure shows that increasing the shearwall stiffness by a factor of 25.5 decreased the deflection at the top by only 35%. This represents an increase in the overall lateral stiffness of the structure by a factor of only 1.5.

A similar behavior has been observed in the earthquake response of frame-shearwall structures. The plots shown in Fig. 12, from a study by Clough and Benuska (14), show the variation of the maximum lateral displacements as well as the girder and shearwall ductility requirements along the height of a 20-story frame-shearwall structure subjected to the first 4 seconds of the 1940 El Centro, Calif., (N-S) earthquake record. The curves shown correspond to structures which differ mainly in the stiffness of the shearwall, the 38-ft. wall structure having a stiffness 25 times that of the structure with the 10-ft. wide wall.

For the particular structures considered, increasing the shearwall stiffness by a factor of 25 decreased the maximum lateral displacement at the top by only 38 percent, the fundamental period of vibration being decreased by about the same percentage. The maximum girder ductility requirements were reduced by about 29%. It is interesting to note the yielding in the upper stories of the more flexible shearwall where changes in stiffness occur.

Interaction forces resulting from differences in deformation patterns under lateral load also arise in coupled or linked shearwalls which have different variations of stiffness over the height of the structure, as shown in Fig. 13. As was noted in the case of frame-shearwall structures, the distribution of the applied lateral load in proportion to the relative lateral stiffnesses of the component elements can lead to serious errors.

ANALYSIS OF FRAME-SHEARWALL STRUCTURES

Whether an analysis is carried out for wind or earthquake loads, the immediate objective is to arrive at a reliable estimate of the response, e.g., forces, displacements or accelerations, of the structure to a particular loading condition or set of conditions.

The determination of the appropriate loading conditions to be used in analysis, with respect to both wind and earthquake loads, is a problem that has only recently been given the attention it deserves (17, 18, 19). For although we are still a long way from designing our structures to specific measures of risk or survival, the need for more relevant data on wind (20) as well as earthquake loads (21) than we presently have is apparent if we are to improve the rational basis of our structural designs. It will be readily granted that without a realistic assessment of the design loads, the use of much of the refinement which has been achieved in analysis would hardly seem justifiable.

Most of the methods of lateral load analysis in common use disregard the dynamic character of the problem. A major reason for this is that all of the presently used building codes specify loads which are to be considered as static loads upon a structure. The recent revisions of some model building codes (13, 24, 32), however, show a recognition of the dynamic character of the problem and attempt to allow for this by incorporating provisions which take account of the period of vibration of a structure. These codes either recommend or require dynamic analyses or experimental studies as a basis for the design of unusual structures or structures susceptible to vibration under wind loading.

Modelling assumptions used in analysis -- The analysis of frame-shearwall structures for the purpose of arriving at preliminary member sizes may be carried out using any of several approximate methods of analysis. These methods, as well as the significance attaching to the various assumptions used are discussed in the recently published report of ACI Committee 442, Response of Buildings to Lateral Forces (25).

The analysis of frame-shearwall structures under lateral load using one of a number of computer programs (9, 12) now available is a fairly straightforward procedure. However, even when using a computer program, it is usually necessary to make some simplifying assumptions in order to arrive at a reasonable idealization of a structure which is amenable to mathematical analysis. It is realized that often, the approximations used in

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analysis are required not only by the relative complexity which a more refined treatment would entail, but are also dictated by the economics of the design fee structure.

I shall try to discuss briefly a few of the more commonly used assumptions in the lateral load analysis of reinforced concrete frame-shearwall structures.

1. Effective width of slab -- The idealization of a structure as a series of linked plane frames may be considered reasonable, as far as the flexural action of the slab is concerned, if the parallel frames are more or less similar so that transverse effects do not produce significant irregularities in the distribution of forces. In such a case, the effective width of slab determines the stiffness of the one-dimensional beam elements used in a plane frame analysis (see Fig. 14).

Analytical and experimental studies (26-29) relating to the problem of "effective slab width" have considered the column-to-panel width ratio as the principal variable and have indicated effective slab width ratios ranging from 0.50 to values greater than unity, the effective width increasing with increasing values of the column-to-panel width ratio. However, it appears that other parameters, such as the slab stiffness, the slab span-to-width (aspect) ratio, the dimension of the column or wall in the plane of the frame relative to the slab span, among others, may also have significant effects on the effective slab width. When adjacent frames differ significantly in the spacing or stiffness of the vertical elements, adjustments may be indicated in the values of both stiffness and carry-over factors of the equivalent beam.

Clearly, the problem of determining the stiffness properties of an "equivalent beam" to properly model the action of a slab for use in a plane frame analysis awaits further study. Here, a distinction may have to be drawn between the criterion of equivalence to be used, depending upon the object of the analysis. Thus, an analysis undertaken to provide values of slab design moments (30) may require different equivalence criteria from an analysis intended to serve as a basis for column design or from an analysis aimed at studying overall frame deformation.

2. Infilled frames -- The behavior of multistory infilled frames, i. e., frames with the space between the members filled with a masonry or cast-in-place concrete wall, under lateral loading is essentially that of a vertical cantilever beam. Although the infill panels in such frames are often

considered "non-structural", full advantage can be made of these by constructing them to ensure their participation in resisting wind loads.

The stiffening effect of the infill panel on the frame can be represented fairly well by a diagonal strut having the same thickness as the panel and an effective width which depends on a number of factors (Fig. 15). Experimental and analytical studies by Smith (31, 32) and by Mainstone (33) indicate that the effective width of the equivalent strut is influenced primarily by the stiffness of the frame elements relative to that of the infill. For the same thickness of infill panel, the effective width of the equivalent strut increases with increasing column stiffness and panel height-to-length ratio and decreases with increasing value of the load and modulus of elasticity of the infill material. The ultimate capacity of an infilled frame is considerably increased by the presence of adjoining panels or other external restraints which effectively increases the stiffness of the frame.

While it may be conservative to disregard the stiffening effect of infilling panels on the behavior of frames subjected to wind loading, neglecting their effect on the response of a structure subjected to earthquakes can be dangerous. Walls filling the space between frame members not only increase the stiffness and hence the frequency of vibration of the structure but also change the mode of deformation under lateral load. This change from a predominantly shear-beam type to a cantilever mode of deformation (Fig. 16) will tend to alter the distribution of lateral forces within a structure when an infilled frame forms a part of a larger structure. Thus, where an infilled frame is connected to open frames, the infilled frame may be expected to be subjected to significantly increased horizontal shears at the base, just as would a shearwall in a typical frame-shearwall interaction (see Fig. 9).

In structures which may be subjected to major earthquakes, infilled frames should be designed as shearwalls, with proper regard to their ductile performance and their interaction with other elements in such structures.

3. Effect of neglecting axial deformation in columns -- Often, particularly in approximate methods of analysis, it is expedient to neglect the axial deformation in the columns of a frame. It is evident that suppressing the axial deformation leads to a stiffer structure. This results in computed values of the lateral displacement and the period of vibration which are less than those which would be obtained if axial deformation is considered. This effect, although relatively small - about 10-15% - for most multistory

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buildings, can be significant for tall slender buildings.

In frame-shearwall structures, neglecting the axial deformation in the columns will tend to throw a greater proportion of the lateral load to the frames, and underestimate the load carried by the shearwall. Also, since the assumption leads to greater axial forces being developed in the outer columns of multibay frames, the beam shears in multibay frames decrease as one moves toward the center of the frame, instead of the other way around, as shown in Fig. 17.

4. Effect of finite width of members; the wide-column model for shearwalls -- A common device used in frame analysis (12) to account for the finite widths of members is to consider that portion of a member from the face to the centerline of the joint as segments possessing infinite flexural stiffness, as shown in Fig. 18. This same device has been used to account for the appreciable depth of shearwalls (34) with satisfactory results as far as the lateral displacements and distribution of lateral load between frame and shearwall are concerned. Where beams frame into shearwalls, the effect of local deformations at the beam-shearwall junction may be allowed for (35) by increasing the span of the flexible portion of the beam by as much as the depth of the beam, lesser adjustments being indicated for shearwall-to-beam depth ratios less than about 6.

When the available frame analysis program does not have the capability of considering the finite width of members as described above, either the center-to-center or the face-to-face distances between frame members will have to be used (see Fig. 19). A study of a limited number of cases indicated that the lateral deflection under static "wind" load based on center-to-center distances can be 30% to 40% greater than deflections obtained considering the finite widths of members. Using the face-to-face distances between joints for member spans leads to lateral deflections only slightly greater than those based on finite member widths.

5. Interaction of gravity and lateral loads; $P-\Delta$ moments -- It is common practice in analysis to consider the effects of gravity loads and lateral loads independently of each other, it being implicitly assumed that the effect of the interaction between gravity loads and the lateral displacements due to lateral loads is of secondary importance. Studies by Majumdar, MacGregor and Adams (36), however, indicate that under statically applied lateral loads, the ultimate capacity of frame-shearwall structures can be reduced by as much as 30% due to the effect of gravity loads acting through the lateral displacements produced by the horizontal loads, or what is com-

monly referred to as the $P-\Delta$ effect (see Fig. 20). A study aimed at determining the variation of the $P-\Delta$ effect with significant design parameters should yield valuable information which may indicate a re-examination of the present safety or load factors relating to wind loads specified in current codes.

In structures subjected to earthquakes, studies by Anderson and Bertero (37) of the dynamic response of multistory unbraced frames indicate that the presence of gravity loads produces early yielding in members with a consequent increase in ductility requirements. In the strong column-weak beam designs intended by current earthquake codes, this increased ductility requirement is confined mainly to the girders. The same study indicated, however, that for frames proportioned to satisfy the usual wind drift criterion, i. e., with a deflection index of about 1/500, the effect of the $P-\Delta$ moments on the inelastic response is negligible. The same observation was made by Goel (38).

Concluding remarks -- The preceding remarks have been confined to but a few of the many aspects of the planning and analysis of frames and frame-shearwall systems. Several other important considerations, such as the two-directional character of the response to wind (18), the interaction of forces resulting from the simultaneous action of the horizontal components of ground motion on inelastic earthquake response (39), the torsional effects of wind and earthquake loading (39, 40), the effect of arbitrarily located openings on the response of shearwalls and effects of temperature and creep, to mention a few, have been looked into only fairly recently. Some of these problems will be discussed by the other authors in this symposium.

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REFERENCES

- (1) Robertson, L. E., "On Tall Buildings" Appendix II, Tall Buildings, Ed. by A. Coull and B. S. Smith, Pergamon Press, 1967.
- (2) Frischmann, W. W. and Prabhu, S. S., "Planning Concepts Using Shear Walls", Tall Buildings, Ed. by A. Coull and B. S. Smith, Pergamon Press, 1967.
- (3) Khan, F. R., "Column-Free Box-Type Framing with and Without Core" 8th Cong. of the Int. Assoc. for Bridge and Structural Engineering., New York, Sept. 1968.
- (4) Anonymous, "Building Design Reduces Steel with Concrete - Tube Wind Bracing", Engineering News-Record, June 3, 1971.
- (5) Goldberg, J. E., "Buckling of Multistory Buildings", Journal of the Eng. Mech. Div. ASCE, Feb. 1965.
- (6) Wright, D. T. and Green, R. "Human Sensitivity to Vibration", Report No. 7, Queen's University, Ontario, Feb. 1959.
- (7) ACI Committee 435, "Allowable Deflection", ACI Journal, Proceedings V. 65, No. 6, June 1968, pp. 433-444.
- (8) "Wind Bracing in Steel Buildings", Transactions, ASCE, V. 105, 1940, pp. 1713-1737.
- (9) Wilson, E. L., "SAP - A General Structural Analysis Program", Structural Engineering Lab., Univ. of California, Berkeley, 1970.
- (10) "EAC/EASE - Elastic Analysis for Structural Engineering" Control Data Corporation, 1970.
- (11) Utku, S., "ELAS - Program for the Analysis of Structures" Dept. of Civil Engineering, Duke University, 1968.
- (12) Derecho, A. T. "Computer Program for the Analysis of Tall Buildings Under Gravity and Lateral Loads", Portland Cement Association, 1971.
- (13) Seismology Committee of the Structural Engineers Associa-

frames and
frame-shearwall
systems

tion of Calif. , "Recommended Lateral Force Requirements and Commentary", 1967 Edition

(14) Clough, R. W. and Benuska, K. L., "FHA Study of Seismic Design Criteria for High-Rise Buildings", HUD TS-3, August 1966, U. S. Dept. of Housing and Urban Development, Federal Housing Administration, Washington, D. C.

(15) Balint, P. S. and Shaw, F. S. "Structural Model of the 'Australia Square' Tower in Sydney", Architectural Science Review, Vol. 8, No. 4, Dec. 1965.

(16) Clark, W. J. and MacGregor, J. G., "Analysis of Reinforced Concrete Shear Wall-Frame Structures", Structural Eng. Report No. 16, Univ. of Alberta, Nov. 1968.

(17) Davenport, A. G., "The Treatment of Wind Loading on Tall Buildings", Tall Buildings, Ed. by Coull and Smith, Pergamon Press, 1967.

(18) Davenport, A. G., "Wind Loading on Tall Buildings", Wind Effects on High-Rise Buildings, Ed. by R. A. Parmelee, Northwestern University, 1970.

(19) Jennings, P. C., Housner, G. W. and Tsai, N. C., "Simulated Earthquake Motions for Design Purposes", Proceedings 4th World Conference on Earthquake Engineering, Chile, 1969.

(20) Singer, I. A., and Smith, M. E., "The Adequacy of Existing Meteorological Data for Evaluating Structural Problems", Wind Loads on Buildings and Structures, Building Science Series 30, National Bureau of Standards, Nov. 1970.

(21) Clough, R. W., "Some Deficiencies in Current Methods of Analysis", Wind Loads on Buildings and Structures, Building Science Series 30, National Bureau of Standards, Nov. 1970.

(22) Uniform Building Code (1970 Edition) Vol. 1, International Conference of Building Officials, Pasadena, Calif.

(23) Cohen, E., and Vellozi, J., "Proposed American Standard Building Code Requirements for Minimum Design Wind Loads", Wind Effects on High-Rise Buildings, Ed. by R. A. Parmelee, Northwestern

response of multistory
concrete structures
to lateral forces

Univ. 1970.

(24) National Building Code of Canada 1970, National Research Council, Ottawa, Canada.

(25) ACI Committee 442, "Response of Buildings to Lateral Forces", ACI Journal, February 1971.

(26) Tsuboi, Y. and Kawaguchi, M., "On Earthquake Resistant Design of Flat Slabs and Concrete Shell Structures", Proceedings 2nd World Conference on Earthquake Engineering, Tokyo, 1960.

(27) Khan, F. R., and Sbarounis, J. A., "Interaction of Shear Walls and Frames", Proceedings ASCE, Vol. 90, ST3, June 1964.

(28) Brotchie, J. F. and Russell, J. J., "Flat Plate Structures", ACI Journal, August 1964.

(29) Carpenter, J., "Flexural Characteristics of Flat Plate Floors in Buildings Subjected to Lateral Loads", Ph. D. Thesis, Dept. of Civil Engineering, Purdue University, August 1965.

(30) Corley, W. G. and Jirsa, J. O., "Equivalent Frame Analysis for Slab Design", ACI Journal, Nov. 1970.

(31) Smith, B. S., "Lateral Stiffness of Infilled Frames", Journal of the Struct'l. Div., ASCE, Dec. 1962, Paper No. 3355.

(32) Smith, B. S. and Carter, C., "A Method of Analysis for Infilled Frames", Proceedings Inst. of Civ. Eng. (London), Vol. 44, Sept. 1969.

(33) Mainstone, R. J., "On the Stiffnesses and Strengths of Infilled Frames", Proceedings Inst. of Civ. Eng. (London), Paper 7360S, 1971.

(34) Weaver, W. Jr. and Oakberg, R. G., "Analysis of Frames with Shearwalls by Finite Elements", Proceedings of Symposium on "Application of Finite Element Methods in Civil Engineering", Nov. 1969, Nashville, Tenn., ASCE.

(35) Michael, D., "The Effect of Local Deformations on the

frames and
frame-shearwall
systems

Elastic Interaction of Cross Walls Coupled by Beams", Tall Buildings, Pergamon Press, 1967.

(36) Majumdar, S. N. G., MacGregor, J. G. and Adams, P. F., "Experimental and Analytical Study of the Behavior of Steel Frame Shear Wall Structures", Structural Eng. Report No. 27, Univ. of Alberta, August 1970.

(37) Anderson, J. C. and Bertero, V. V., "Seismic Behavior of Multistory Frames Designed by Different Philosophies", Earthquake Eng. Res. Center Report No. EERC 69-11, Univ. of Calif., Berkeley, Oct. 1969.

(38) Goel, S. C., "P- Δ and Axial Column Deformation in Aseismic Frames", Journal of the Struct'l. Div., ASCE, August 1969, Paper No. 6738.

(39) Nigam, N. C. and Housner, G. W., "Elastic and Inelastic Response of Frame Structures During Earthquakes", Proc. 4th World Conf. on Earthquake Eng., Chile, Jan. 1969, Vol. II.

(40) Koh, T., Takase, H. and Tsugawa, T., "Torsional Problems in Design of High-Rise Buildings", Proc. 4th World Conf. on Earthquake Eng., Chile 1969, Vol. II.

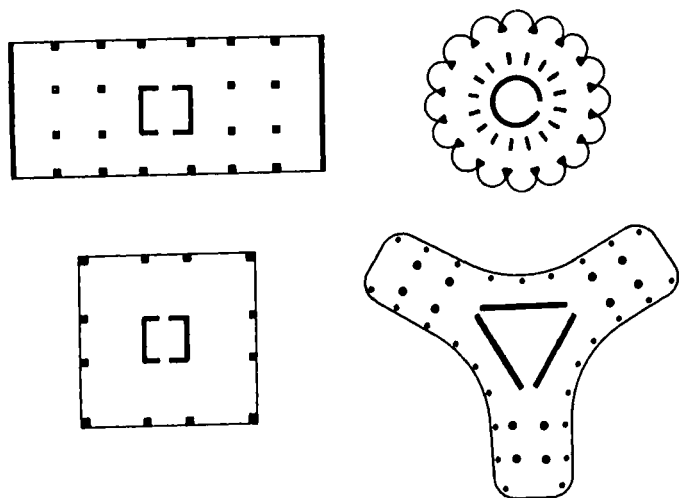


Fig. 1. Typical
plans - frame -
shear wall
structures

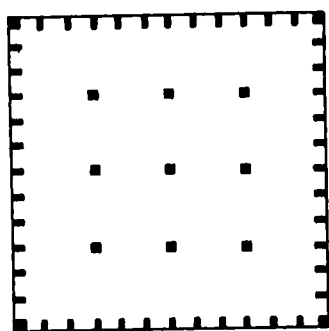


Fig. 2. Framed tube

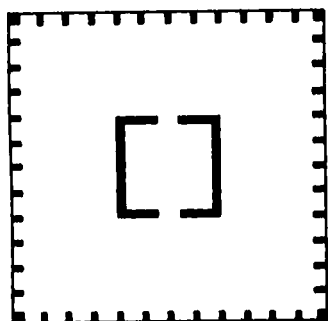


Fig. 3. (left) framed tube with core; (right) perspective view of a building using the framed-tube-with-core system

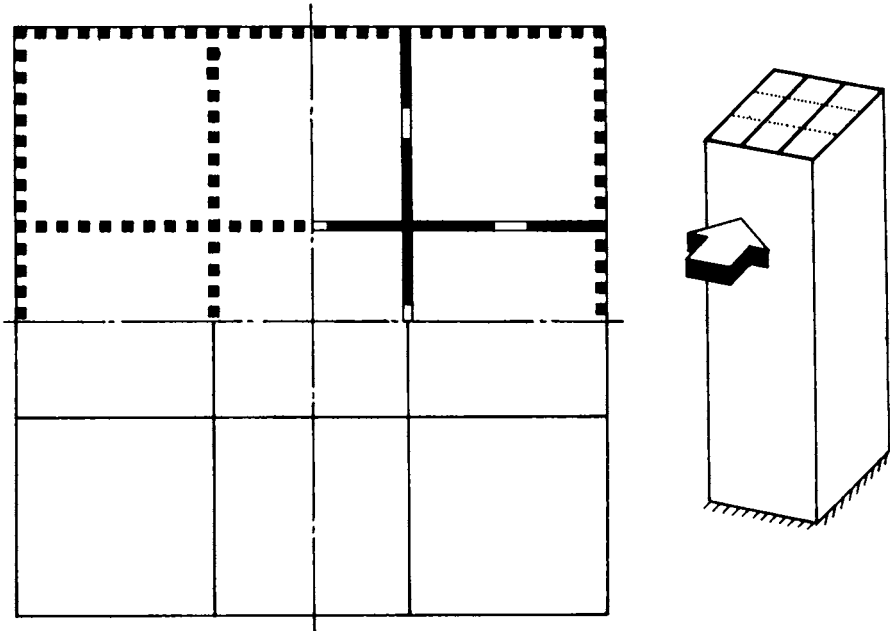


Fig. 4.

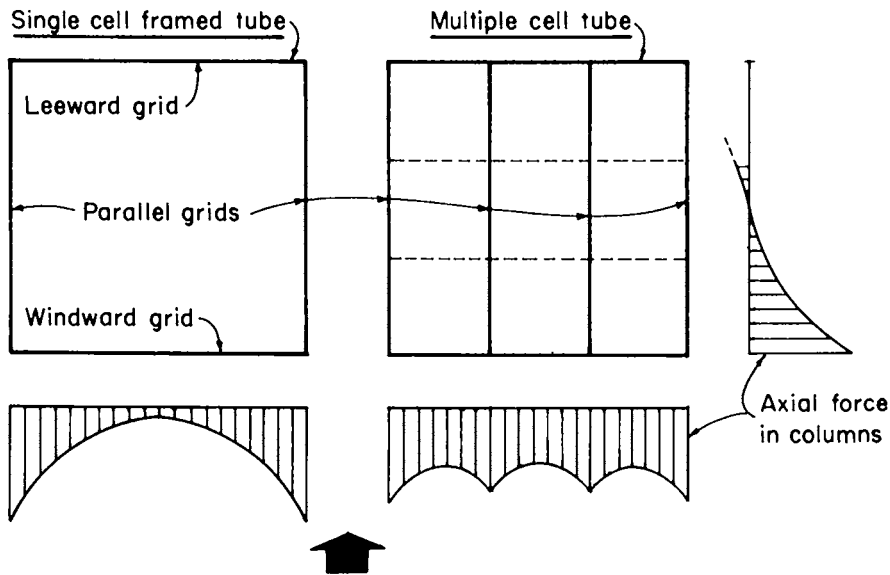


Fig. 5.

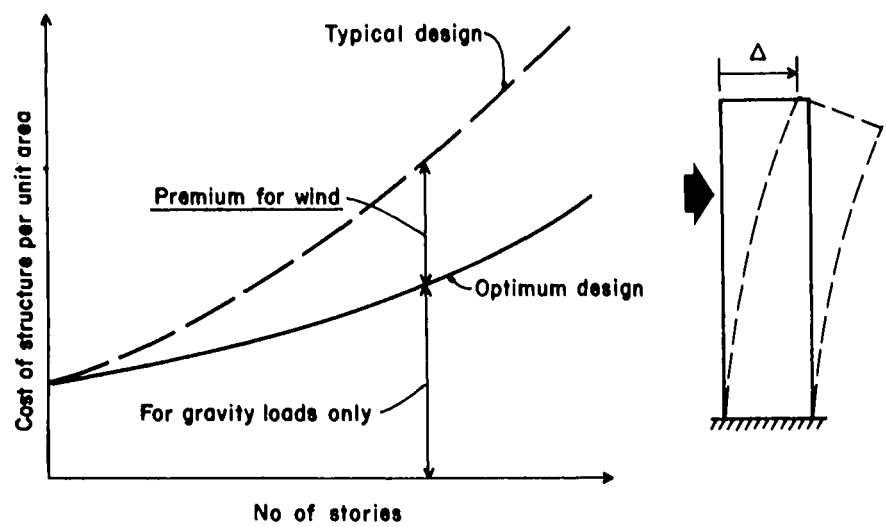


Fig. 6.

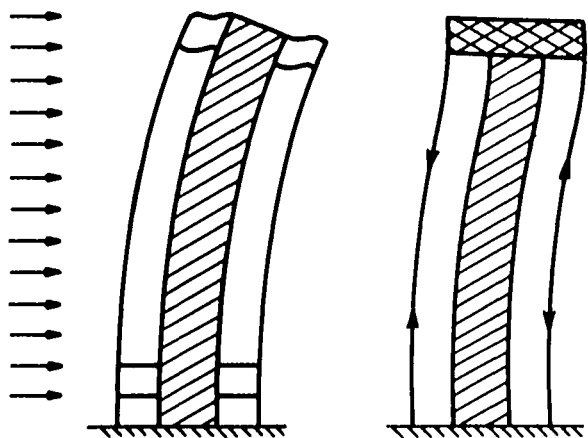


Fig. 7. Tying core with exterior columns

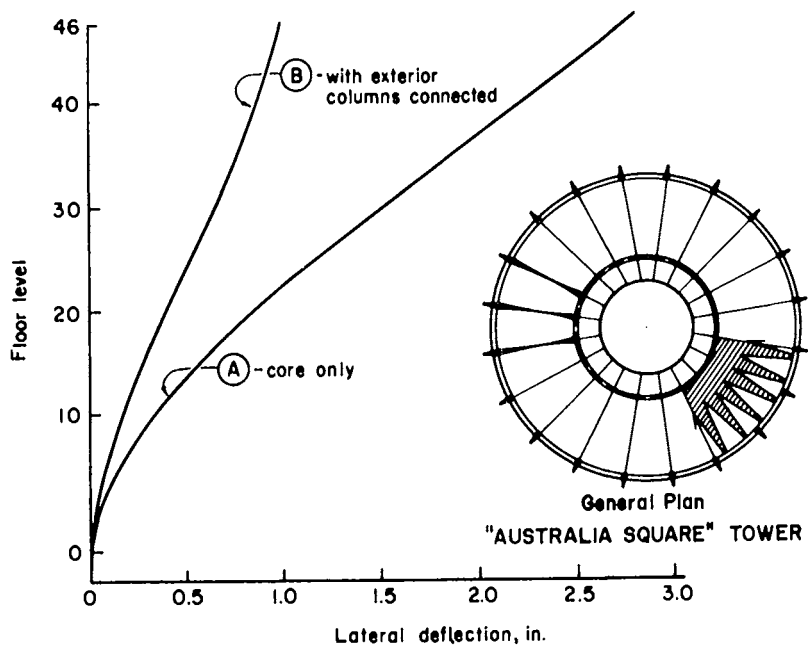


Fig. 8.

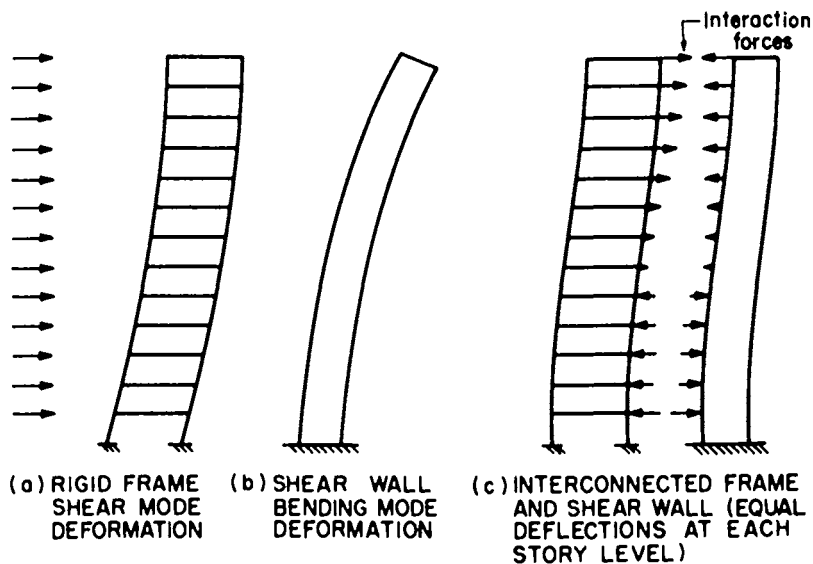


Fig. 9.

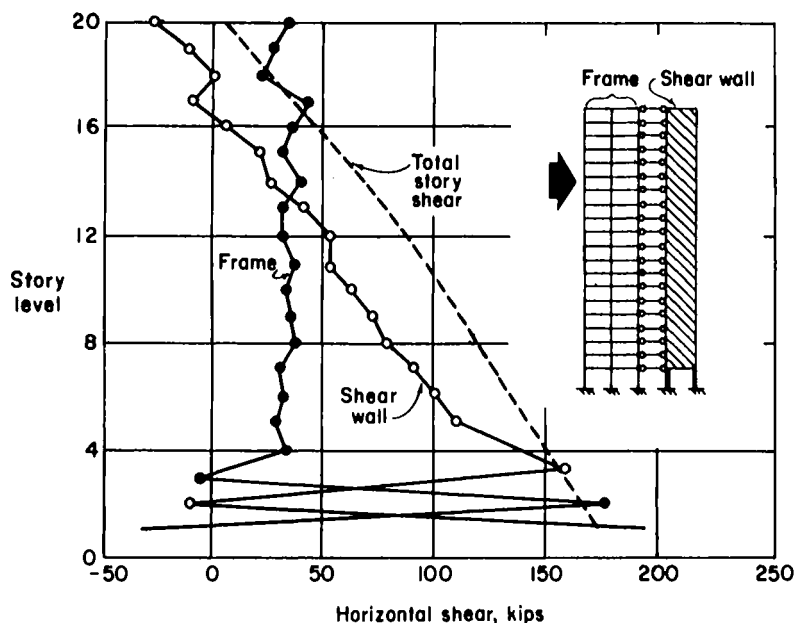


Fig. 10. Distribution of applied horizontal load between frame and shear wall

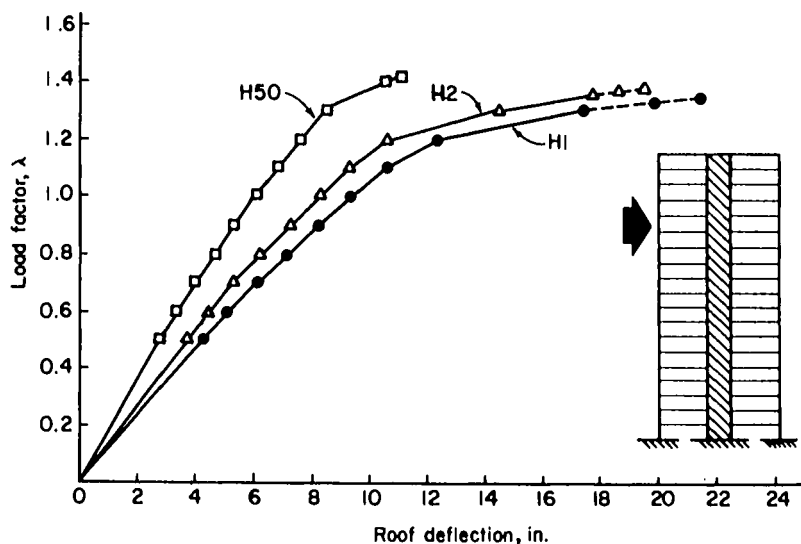


Fig. 11. Effect of increasing shear wall stiffness (from ref.16)

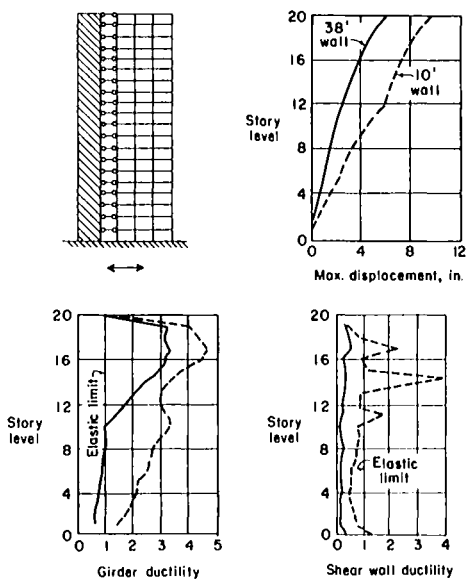


Fig. 12. (from ref. 14)

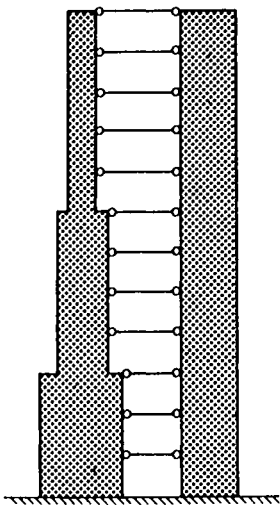


Fig. 13.

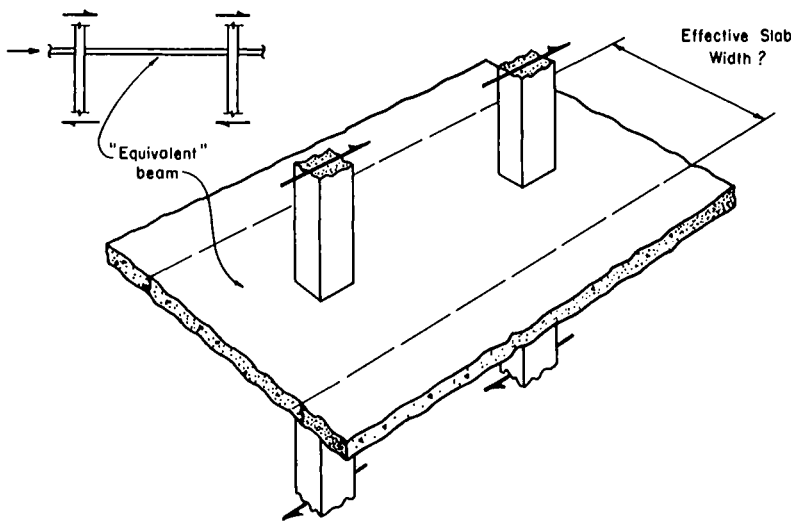


Fig. 14.

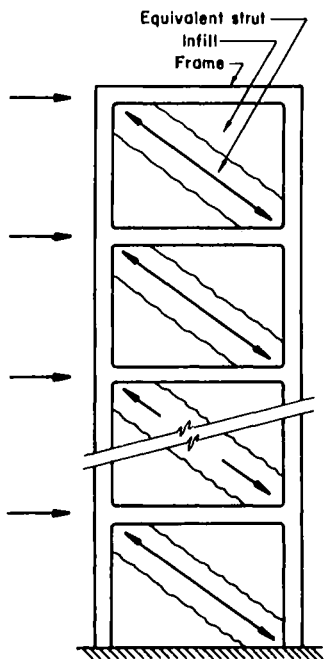


Fig. 15.

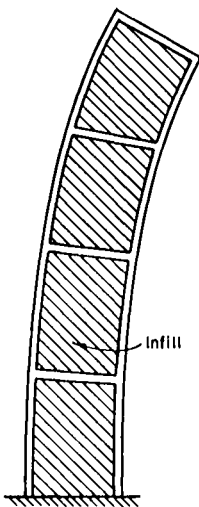
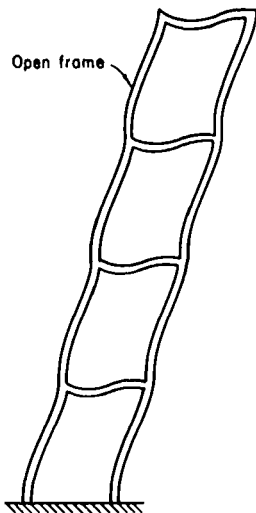


Fig. 16.

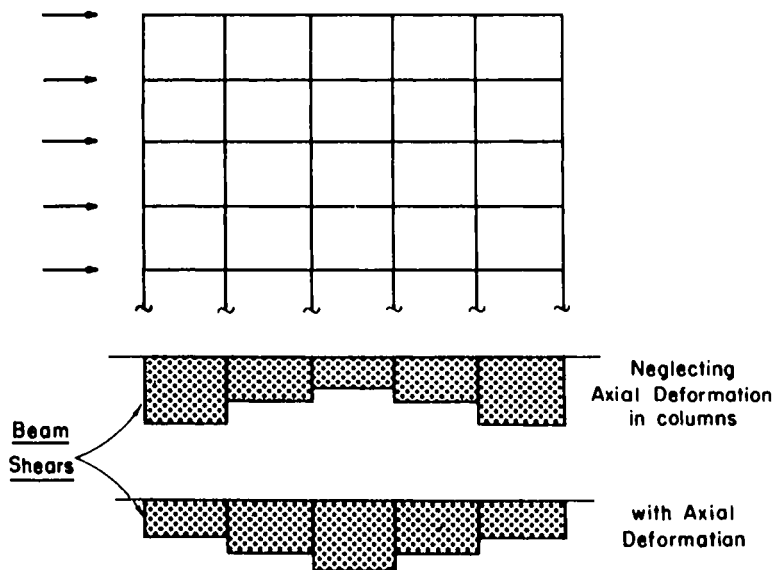


Fig. 17.

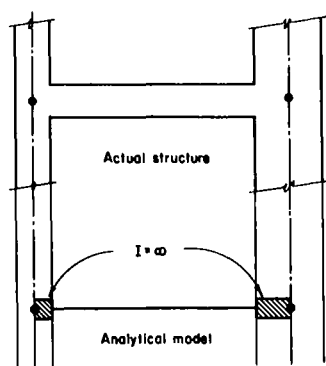


Fig. 18. Modeling finite width of members

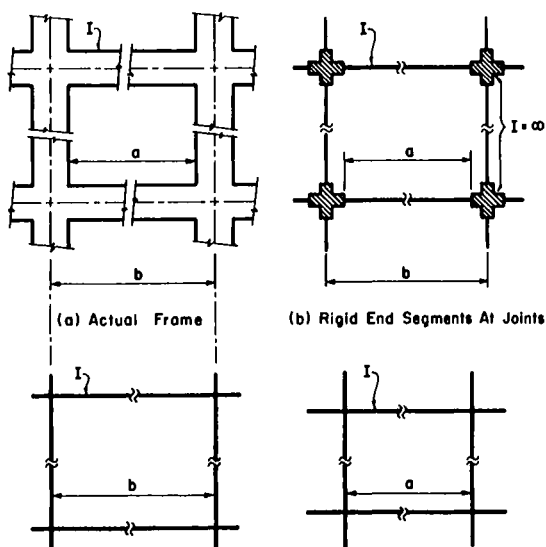


Fig. 19.

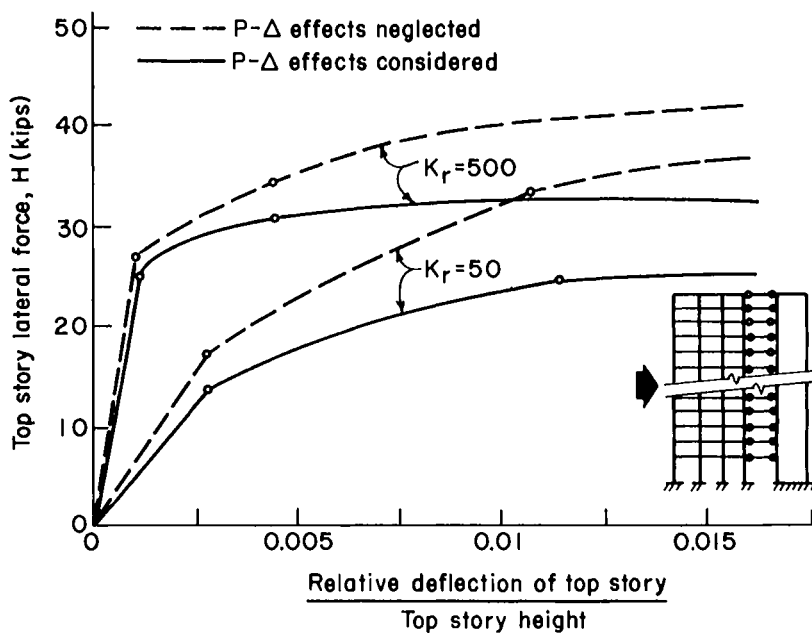


Fig. 20. (from ref. 36)

PAPER SP36-3

ANALYSIS AND DESIGN OF FRAMED TUBE STRUCTURES FOR TALL CONCRETE BUILDINGS

BY
FAZLUR R. KHAN
NAVINCHANDRA R. AMIN

Synopsis: The behavior of framed tube structures is discussed from an overall structural system point of view. The influence of various structural parameters affecting tubular behavior is emphasized. The concept of the equivalent reduced plane frame modeling technique is used for developing a series of "influence" curves for the preliminary analysis and design. An example problem is worked out using these curves and the results are compared with a more exact solution of a computer analysis.

Keywords: axial loads; beam-column frame; beams (supports); columns (supports); deflection; framed-tube buildings; high rise buildings; lateral pressure; moment distribution; multistory buildings; reinforced concrete; shear properties; shear walls; spandrel beams; stiffness; structural analysis; structural design; tube-in-tube buildings; webs (supports); wind pressure.

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INTRODUCTION

It is only in the last 20 years that reinforced concrete has found increasing use in the construction of tall buildings. In its initial development in the early 1900's reinforced concrete buildings were limited to only a few stories in height. The structural type used was the traditional beam-column frame system which made the construction of taller buildings relatively expensive and therefore, economically unfeasible. In the early 1950's the introduction of shear wall type of construction opened up the possibility to use concrete in apartment and office buildings as high as 30 stories. Taller buildings still remained economically unattractive, and technically inadequate, because the shear walls which were mostly used in the core of the building were relatively small in dimensions compared to the height of such buildings, leading to insufficient stiffness to resist lateral loads. It was obvious that the overall dimensions of the interior cores were too small to economically provide the stability and stiffness for buildings over 30 to 40 stories. The natural tendency then was to find new systems that would utilize the perimeter configurations of such buildings rather than to rely on the core configurations alone. The development of the framed tube system was, therefore, a logical outcome of this challenge. The framed tube system in its simplest form consists of closely spaced exterior columns tied at each floor level with relatively deep spandrel beams, thereby creating the effect of a hollow concrete tube perforated by openings for the windows. Since the system simulated a hollow tube using perimeter closely spaced frame elements, it is referred to as "framed tube" (Fig. 1). This system was apparently first applied on the design of the 43-story DeWitt-Chestnut apartment building in Chicago in 1963 (Fig. 2). Since then this system has received wide acceptance among designers all over the world, and many variations are being used in a number of buildings under construction.

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From the point of view of construction economy, the framed tube compares favorably with the normal shear wall type of construction for medium-rise buildings, but provides a distinct economic advantage for taller buildings. Moreover, the closely spaced column system has the additional advantage of also being the window wall system, thus replacing the vertical mullions for the support of the glass windows. In some recent buildings the elimination of the traditional curtain wall with its metallic mullions was in itself the justification for choosing this structural system.

The center-to-center spacing of the exterior columns in the framed tube structural system is generally from 4 ft. to a maximum of about 10 ft. Depending on the overall proportion and height of the building, the maximum center-to-center spacing of the peripheral columns can probably be increased to 15 ft. The spandrel beams interconnecting the closely spaced columns generally vary from 2 ft. in depth to about 4 ft. in depth with widths from 10 inches up to 3 ft. In designing the framed tube structural system it is necessary to keep the proper balance of stiffness between the spandrels and the columns so that both of these elements are efficiently utilized to provide stiffness of the structure against lateral sway, and to assure the overall strength of the tube system to resist lateral forces. In most recent structures stiffness for limiting lateral sway controlled the proportions more often than strength requirements. It is obviously advisable to optimize the sub-system represented by two columns and two spandrels as shown in Fig. 3 to arrive at the optimum spacing of the columns and the proportion of the spandrels and the columns in terms of overall architectural program. In the DeWitt-Chestnut apartment building the columns were spaced on 5 ft. 6 in. centers and the spandrels were 2 ft. deep. The spacing of the columns in this case was also related to the module for interior planning of the apartment floors.

The framed tube structural system has expanded in its application and in its variations over the last five years. One of these variations is its application with an interior shear wall, commonly referred to as "tube in tube" system, as used in the 52-story One Shell Plaza Building in Houston, reaching a height of 714 ft. Another variation of the framed tube system has been used as the exterior envelope in conjunction with the traditional steel framing for the interior of the building. This system, known as the SOM Composite System, has been used in three major tall buildings in the country, namely: the 24-story CDC Building in Houston, the 35-story Union Station Building in Chicago, and the 50-story One Shell Square Building in New Orleans.

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In view of the wide and varied application of the framed tube system, there is an obvious need for developing a preliminary analysis and design method for such a system. Of course, with availability of large computer programs, the final solution may be easily refined; however, the use of computers for preliminary design of even a moderate size framed tube may prove too expensive. Therefore, a method of preliminary analysis and design (using "influence curves") for a wide range of proportions of the framed tube elements and geometry is presented.

This paper is primarily intended for a clearer and better understanding of the behavior of the framed tube system.

BEHAVIOR OF THE FRAMED TUBE SYSTEM

The framed tube system combines the behavior of a true cantilever, such as a shear wall, with that of a beam-column-frame. The overturning under lateral load is resisted by the tube form causing compression and tension in the columns, while the shear from the lateral load is resisted by bending in columns and beams primarily in the two sides of the building parallel to the direction of the lateral load as shown in Fig. 4. Therefore, for all practical purposes the bending moments in these columns can be determined by judicious choice of the point of contraflexure in each story. While it is true that in the lower few stories, as well as in the upper few stories, the point of contraflexure does not remain in the middle of story height, the intermediate stories which constitute the major portion of the building generally have the point of contraflexure at midheight of each story. It is, therefore, possible to compute the bending moments in these columns with reasonable accuracy for any known lateral shear at each story. One can, of course, make a simple iterative "Maney-Goldberg" type slope-deflection solution, or a modified moment distribution solution to determine more accurate moments in these columns. In fact, such an iterative solution will also give a good approximation of that portion of the total deflection which is caused by the frame action only. To this the additional overturning deflection caused by tension or compression in the column must be added to compute the total lateral deflection.

The cantilever tube type behavior becomes significant when the overturning of the entire building due to lateral load is considered. For analyzing the overturning of the entire framed tube, the exterior column system can be considered as part of a rigidly diaphragmed hollow tube. However, in recognition of the fact that the webs of the hollow tube, that is, the two sides parallel to the direction of the lateral force are not truly solid webs

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but are, in fact, grid frames, one must consider then the effect of loss of efficiency due to the flexibility of this "web-frame" causing what is known as "shear lag", as shown in Fig. 4. For a very preliminary estimate of the overall resistance, as well as the deflection of the building, the effective configuration of the "tube" could be reduced to two equivalent channels resisting the total overturning moments (Fig. 5). Experience has indicated that for preliminary designs "channel flanges" normally should not be more than half the depth of the web (walls parallel to lateral load), or more than about 10% of the height of the building. These approximate rules have generally given conservative values of shear and moment as compared to the actual forces in the exterior columns obtained by the exact analysis performed subsequently by a generalized computer program such as STRESS, STRUDL, or others.

INITIAL PRELIMINARY DESIGN APPROACH

The overturning moment resisted by the two equivalent channels will produce axial forces in the closely spaced columns of these channels, as well as shear forces in the connecting spandrels. The preliminary estimate of the axial forces in the columns as well as the shears in the connecting spandrel can be based on the classical beam theory and can be expressed as follows:

$$P_w = \frac{M \times C \times A_c}{I_e}$$

and

$$V_s = \frac{V_w \times Q \times h}{I_e}$$

where

P_w = Axial force due to wind

M = Overturning moment

I_e = Effective moment of inertia of the tube

V_s = Spandrel shear

V_w = Total wind shear

Q = Sum of first moment of column areas about the neutral axis

C = Distance of any column from the neutral axis

h = Story height

A_c = Cross-sectional area of column

From the structural point of view the Framed Tube differs from the solid tube by the many large openings. Its behavior is, therefore, of a

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hybrid nature showing characteristics of the pure frame as well as of the pure tube. Even though one may expect the Framed Tube behavior to be more like a tube resisting the lateral forces through axial forces in the columns, significant moments develop in the columns in the two walls parallel to the wind direction.

In a Framed Tube type structure the preliminary design method will generally indicate a reasonably uniform shear force in the spandrel beams along the two exterior walls parallel to the wind force. The preliminary moments in the spandrel beams are consequently derived from these shears. The preliminary design of the closely spaced columns should be based on the known dead and live loads added to the axial forces due to overturning; the moments caused by story shears should also be considered.

PRELIMINARY DESIGN USING INFLUENCE CURVES

To achieve a more accurate design than the equivalent channel method proposed earlier for framed tube structures of any proportion and height within practical range, the authors developed influence curves which can be directly used for a relatively accurate preliminary design. These curves have been developed on the basis of a number of computer runs on a ten-story equivalent framed tube with variable nondimensional parameters representing ratios of shear stiffness S_b of the spandrel beam to the axial stiffness S_c of the columns, and a linearly varying ratio of bending stiffnesses of columns to spandrels. A check of a number of framed tube buildings showed that this ratio ranged from 0.95 at the roof to 0.4 at the ground level. For this paper a ratio of 0.75 at the roof to 0.5 at ground level is assumed. A separate computer run, with a constant ratio of bending stiffness of columns to spandrels of 1.0 did not change the qualitative results.

The significant structural properties affecting the tube action are:

1. Bending stiffness:

$$K_c \text{ for column} = \frac{I_c}{H}$$

$$K_b \text{ for spandrel beam} = \frac{I_b}{L}$$

2. Shear stiffness of the spandrel beams (defined as the force required to displace one end of the spandrel a unit distance at right angles to the axis of the beam):

$$S_b = \frac{12 EI_b}{L^3}$$

3. Axial stiffness of the column (defined as the axial force required to shorten the column a unit distance along the axis of the column):

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$$S_c = \frac{A_c E}{H}$$

where

- I_c = moment of inertia of the column
- I_b = moment of inertia of the spandrel beam
- A_c = cross-sectional area of the column
- H = height of column
- L = effective span of the spandrel beam
- E = modulus of elasticity

the controlling parameters of framed tubes are:

$$\begin{aligned} \text{Stiffness Ratio} &= \frac{K_c}{K_b} \\ \text{Stiffness Factor } S_f &= \frac{S_b}{S_c} \\ \text{Aspect Ratio } R &= \frac{\text{Flange frame}}{\text{Web frame}} \end{aligned}$$

To plot influence curves framed tubes have been analyzed for uniform lateral load and for the following ranges of variables: Aspect Ratio values of 0.5, 0.666, 1.0, 1.5, and 2.0; Stiffness Factor values of 0.1, 1.0, 10.0, and a linearly varying Stiffness Ratio of 0.75 at roof and 0.5 at ground level for all values of Aspect Ratio and Stiffness Factor combinations.

Analysis of the equivalent framed tube from which the design curves have been developed was based on the configuration shown in Fig. 6, and direct solutions were obtained by converting the framed tubes into equivalent plane frames, as shown in Fig. 7.

USE OF NONDIMENSIONAL CURVES FOR PRELIMINARY DESIGN

A total of nine nondimensional preliminary design curves, Figs. 8 to 16 are presented. These curves are primarily for computing column axial force coefficients for flange and web frame columns and shear force coefficients for the web frame beams. All these coefficients relate to unit values for the corresponding forces of the ideal tube. The main purpose of developing the curves was to provide the design engineer with a tool to determine the tubular characteristics of any given framed tube and quickly compute the total deflection and bending moments and shears in the beams caused by the tubular nature of the entire structure. To make all the curves applicable to a wide range of realistic proportions of actual foreseeable buildings, these curves have been plotted against nondimensional parameters representing the basic properties of the column and beam elements and Aspect Ratios.

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To make the curves usable for any number of stories as well as any number of columns around the perimeter of a building, reduction model techniques similar to those used by the first author in previous developments (1, 2, 3) have been used and are described in the following sections:

REDUCTION MODELING APPROACH

To achieve uniformity of reference to the design curves, computer solutions were obtained on a number of specific ten-story frame-tubes, each representing Stiffness Factor, Aspect Ratios and Stiffness Ratio indicated on the curves. Although the solutions were obtained on 10-story hypothetical framed tubes, it can be shown that the tubular behavior of any framed tube of any number of stories can be simulated by converting it into an equivalent framed tube of the same height, but having less or more number of stories. For any given plan proportion commonly referred to as the Aspect Ratio "R" of a building, the variables that directly affect the tubular stress distribution are the shear stiffness S_b of the spandrel beams and the Axial Stiffness S_c of the columns. For reasonably uniform spacing of perimeter columns S_b and S_c in the graphs represent the sum total of all columns and beams around the perimeter. If, however, Stiffness Factor S_f varies on each face, an average value can be taken for a reasonable approximation.

It can be shown that any framed tube of "N" stories can be reduced to a 10-story equivalent tube of the same height by considering a transformation of the actual stiffness Factor to a 10-story equivalent Stiffness Factor of S'_{f10} where $S'_{f10} = S_f \times (N/10)^2$.

Using this simple reduction model technique, any given actual framed tube can first be converted to an equivalent 10-story framed tube and then analyzed by using the "influence" curves.

TOTAL BEHAVIOR OF A FRAMED TUBE

As was pointed out earlier in this paper, the framed tube system always has two components of its behavior: (a) the frame action of the two sides parallel to the direction of the lateral load, and (b) the overturning action of the entire tube causing only tension and compression in the exterior columns. All the "influence" curves presented in this chapter are for the evaluation of the tube action only, although the use of these curves will allow one to compute the approximate moments and shears in the spandrels which will define one boundary of the column moments. The other boundary of the column moments at any typical story should be obtained by assuming the

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point of contraflexure at mid-height of each story. For preliminary design the higher of the two values should be used.

To compute the total deflection, the additional deflection due to frame action must be calculated separately and added to the deflection caused by tube action.

EXAMPLE PROBLEM

To illustrate the use of "influence" curves for preliminary designs, a hypothetical framed tube 50 stories high and 130 x 180 feet in plan with columns at 10 feet on center and a 13 feet floor-to-floor height is considered.

Before using the "influence" curves, the basic parameters of the structure, like Aspect Ratio, average Stiffness Factor, average Stiffness Ratio, and frame deflection, must be computed. Also, the deflection and forces in columns and beams for an ideal tube of the same Aspect Ratio must be known. Basic section properties and parameters are presented in Tables 1 and 2.

Frame deflection can be computed by any standard method and is approximately 3.79 inches. From Table 2, by conjugate beam method cantilever deflection is computed as 7.5 inches. Any other similar method could also give reliable results.

The actual forces in columns, shears in beams and deflection of the system, can now be computed using the "influence" curves. For this example, Fig. 10, SF = 10 curve is used for computing the column axial forces in first story columns, and Fig. 15(c), SF = 10 curve is used for computing the beam shears in the first story beams of a 10-story model. The above results are summarized in Table 3 and compared with the results obtained from a more exact computer solution.

The deflection of the total system is the sum of the frame deflection plus the ideal tube deflection times the magnification factor. This magnification factor is defined as the ratio of the sum of the ideal tube column forces to the actual column forces of the flange columns. For this example the sum of the ideal tube column forces is $1280 \times 10 = 12800$ kips and the sum of the actual column forces is 10825K. Hence, the magnification factor is 1.17.

$$\Delta \text{ Total} = 3.79 + 7.5 \times 1.17 = 12.49$$

$$\Delta \text{ Actual} = 11.51''$$

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Comparing the values of the above example problem with an exact computer analysis shows that the use of "influence lines" gives fairly reliable results with an accuracy of 90 to 95 percent. The shear forces in the spandrel beams of the web frames (parallel to the wind direction) differ by 5% only from the exact analysis at the center of the frame; the difference, increasing progressively towards the corners. However, since in a typical framed tube structure the size of the spandrel beam is generally constant at any level, the inaccuracy of the shears in the corner spandrel beam during the preliminary design is of little consequence.

CONCLUSION

A framed tube type of structural system is used increasingly for tall concrete as well as steel buildings. While large capacity computers can be used to finalize the design of such structures, they cannot substitute the improved understanding of such systems. Furthermore, the use of computers for preliminary design of even a moderate size framed tube may prove too expensive. Some simple preliminary design methods have therefore been presented in this paper. It is hoped that the "influence" curves which can be used for most practical ranges of framed tubes will not only help save time at the preliminary design stage, but also provide a clearer understanding of the general behavior of the framed tube system.

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References

- (1) "Current Trends in Concrete High Rise Buildings" by F. R. Khan. Proceedings, Symposium on Tall Buildings, University of Southampton, England, April, 1966.
- (2) "On Some Special Problems of Analysis and Design of Shear Wall Structures" by F. R. Khan. Proceedings, Symposium on Tall Buildings, University of Southampton, England, April, 1966.

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- (3) "Tendances Actuelles dans la Construction des Immeubles d Grande Hauteur a Structure en Beton Arme et en Acier" by F. R. Khan, Annales de l'Institut Technique du Batiment et des Travaux Publics, Supplement au No. 281, May 1971.

TABLE 1

Computations of Basic Parameters for Equivalent Ten-Story Model

Floor	Column Properties			Spandrel Beam Properties			Stiffness Factor $S_f = S_b / S_c$	$S_{f10} = S_f(N/10)^2$	Stiffness Ratio = K_c / K_b
	Area In^2	M. of I. In^4	Axial Stiffness $S_c K / \text{In}$	Area In^2	M. of I. In^4	Shear Stiffness $S_b K / \text{In}$			
1 - 5	972	462000	24400	1455	452000	13000	0.53	13.2	0.795
6 - 10	810	447000	20400	1200	372000	10700	0.525	13.1	0.925
11 - 15	810	447000	20400	1200	372000	10700	0.525	13.1	0.925
16 - 20	810	447000	20400	1200	372000	10700	0.525	13.1	0.925
21 - 25	717	400000	18000	1058	330000	9520	0.525	13.1	0.935
26 - 30	615	363000	15400	940	282000	8140	0.52	13.0	1.0
31 - 35	615	363000	15400	940	282000	8140	0.52	13.0	1.0
36 - 40	510	307000	12800	754	234000	6750	0.52	13.0	1.01
41 - 45	510	307000	12800	754	234000	6750	0.52	13.0	1.01
46 - 50	510	307000	12800	754	234000	6750	0.52	13.0	1.01

Average Stiffness Factor for ten-story model = 13.05

Average Stiffness Ratio = 0.95 Aspect Ratio = $\frac{180}{130} = 1.4$ For this example use Stiffness Factor = 10.0
and Aspect Ratio = 1.5

TABLE 2

Computations for Ideal Tube

Floor	1	5	10	15	20	25	30	35	40	45
Overturning Moment Ft-Kip	975000	789750	624000	477750	351000	243750	156000	87750	39000	9750
Tubular Moment Of Inertia = $\sum A R^2 \text{ Ft}^4$	333444	277870	277870	277870	245966	210975	210975	174955	174955	174955

Column axial force = $\frac{M \times C}{I} \times A_c = 1280^k$
at Level 1Spandrel beam shear at center line spandrel = $\frac{VQ}{I} \times H = 2850^k$

TABLE 3
Comparison of Actual and Calculated Forces
in Columns and Spandrel Beams

Flange Column Axial Forces			Shear Force in Web Frame Beams		
Col. Location	Using Influence Curves Kips	Actual Kips	Beam Location	Using Influence Curves Kips	Actual Kips
0	1660	1583	1	2340	1954
1	1530	1426	2	2620	2257
2	1280	1237	3	2750	2450
3	1150	1100	4	2810	2565
4	1020	1001	5	2850	2640
5	900	929	6	2850	2690
6	830	877	7	2850	2700
7	830	841			
8	830	818			
9	795	806			