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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.

response of multistory concrete structures to lateral forces

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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 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 lineal or if they are not of a uniform height or cross-section, this seemingly simple process becomes a major problem.

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

what do we need to know about the behavior of structural concrete shear wall structures

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:

$v = .05\, f'_c = 200$ psi; v_u = approximately 400 psi

1967 Uniform Building Code:

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

1971 American Concrete Institute Building Code:

$v_u = 10\, \sqrt{f'_c} = 630$ 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 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-joist 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).

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.

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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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

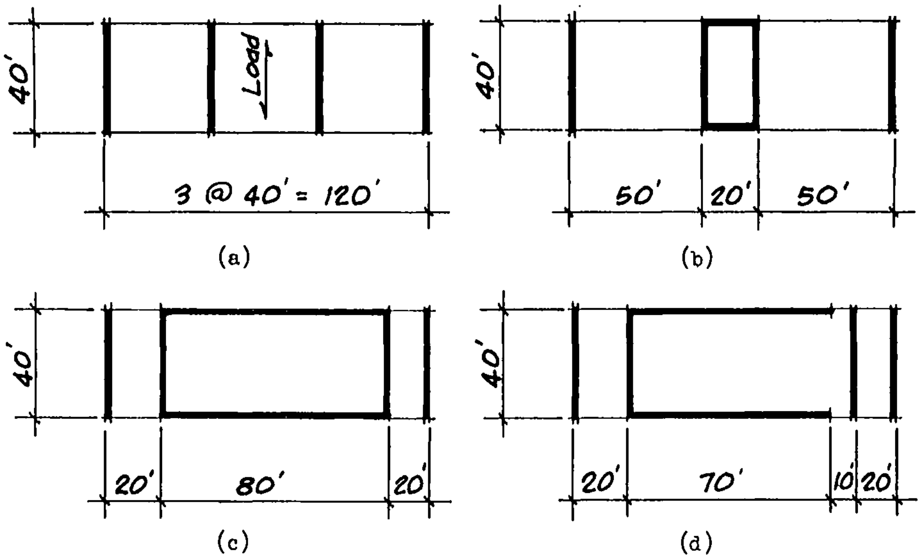

Fig. 1. Non-lineal shear walls-rigid concrete diaphragm in all cases-walls same height and same thickness

WALL	CONNECTING BEAM	Rigidity of System
1	Solid 40' Wall No opening	100
2	12' 18" 24" 12" $I = 1.63\ FT^4$	92
3	24" 12" $I = 0.67\ FT^4$	92
4	15' 18" $I = 0.37\ FT^4$	91
5	10' 18" $I = 0.25\ FT^4$	87
6	12" 18" $I = 0.025\ FT^4$	50
7	2 Walls @ 18' No Beams $I = 0$	18

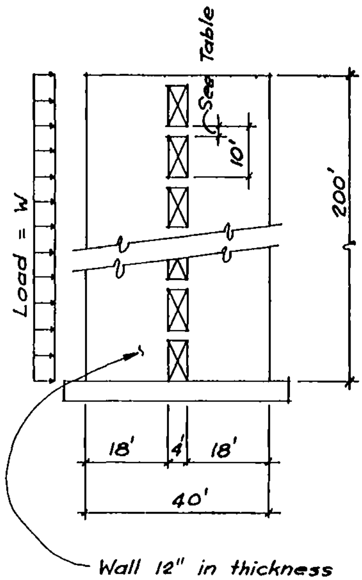

Fig. 2. Coupled walls-connecting beam stiffness vs. rigidity of system