EFFECTIVE STIFFNESS AND CARRY OVER FACTORS FOR FLAT PLATES - MOIRÉ MODEL STUDY

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by

R. Nantasarn

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PREFACE

This thesis was carried out with the intent of determining the effective stiffness and carry-over factors of plates transversely loaded by columns or shear walls. Four plexiglas models were tested by R. Nantasarn and two micro concrete models were tested by A. Parnichkul. The test results were then compared with each other and published test results.

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CHAPTER I

INTRODUCTION

1.1 Object and Scope. In recent years the use of multistory buildings has grown rapidly for both residential and commercial purposes. As buildings increase in height, it becomes increasingly important to ensure adequate lateral stiffness against wind, seismic or blast loads. A commonly used type of structure which provides this lateral stiffness consists of a central core, which acts as a shear wall and accommodates elevators, stairs and services, surrounded by peripheral and possibly interior columns. The floor systems generally consist of flat plates, flat slabs, waffle slabs or concrete joints.

In the lateral load analysis of such shear wall-frame structures, the structure is often idealized as a series of parallel planar frames for which the "beams" are portions of the floor systems. One of the most complex problems in the analysis is the estimation of the effectiveness of the floor systems in resisting the relations of the shear walls and columns. In the past, it has sometimes been assumed for analysis purposes that the frame carries only vertical loads while shear walls resist all lateral loads. If, however, the shear wall-frame interaction is considered, the frame and shear walls tend to prevent each other from taking their natural free deflected shapes and a redistribution of forces among them will result. The calculated stiffness of the structure will also be increased. The force distribution in the structure depends on the stiffness of the members connecting the frame and the shear walls.

For flat plate shear wall structures, the stiffnesses of the connecting members depend on the widths of the plates that are effective in resisting lateral loads.

The object of this study was to measure experimentally the stiffness and carry-over factors for flat plates loaded by couples produced by loads applied transversely to columns and shear walls of various dimensions and shapes. The stiffness factors are used to compute effective slab widths for the various type of structure. The effective slab width is the width of a rectangular beam that would have the same flexural stiffness at a laterally loaded column or shear wall as the actual flat plate.

Four 1/24 scale plexiglas and two 1/16 scale micro concrete flat plate models were used in the tests. Each model consisted of several continuous flat plate panels supported by peripheral columns and loaded by couples applied transversely either to central column or shear wall stubs or to peripheral column stubs.

1.2 Relationship to Previous Tests. Interaction of wideplates with laterally loaded columns and shear walls has been treated analytically and experimentally by several investigators. Tsuboi and Kawaguchi [(1)] carried out tests on nine mortar plate models loaded through column stubs, and compared their test results with those from an elastic finite difference analysis. They reported effective plate widths based on stiffness factors for interior columns.

Khan and Sbarounis,[(2)] investigating the "Interaction of Shear Wall and Frames" reported an approximate solution obtained from simple mathematical and physical models. The analytical study was supported by a sypli-fied elastic model test showing good agreement. Also the results of their study of column plate interaction were in close agreement with those of Tsuboi and Kawaguchi.

Brochie and Russel[3] who studied an entire flat plate structure presented an analytical study (assuming elastic behavior) of flat plates subjected couples produced by lateral loads acting on the columns. They assumed the plate to be supported on an elastic medium with a variable modulus of elasticity and the loads were assumed applied directly to the plate. They expressed their findings in terms of effective plate widths.

Carpenter[4] studied the subject of effective plate stiffness and reported effective slab widths which were in good agreement with those obtained by Brochie and Russel, although they were based on different definitions of stiffness. While Carpenter used the moment rotation characteristic of one column with other columns remaining fixed, Brochie rotated simulteneously every column on the axis of rotation and obtained a somewhat lower stiffness due to the transverse carry-over factor.

Bernard and Schweighofer [5] studied plexiglas models subjected to lateral loads. Each model consisted of two shear walls connected by a series of plates of varying sizes. Test results showed a good agreement with an analytical study employing Rosman's formula. When the width of the plates was increased, different stress values resulted. It was reported that the difference could be caused by stress concentrations due to unconnected shear walls. The tests verified that within the elastic rang the entire width of the prototype plate could be considered as effective stiffness for frame action for two coupled shear walls. Carry-over factors were not considered in this study.

The ACI Building Code[6] 318 - 63 does not recommended specific values for effective stiffness, but does imply in Sec. 2102(g), the participation of the entire panel width for transfering unbalanced moment. The pertinent section states: "When concerning with lateral loads, the moment shall be distributed between the column and middle strips in the same proportions as prescribed for negative moments due to gravity loads." A further statement in the same section reads in part, "A slab width between lines that are 1.5t each side of the column may be considered effective."

Distasio and Van Buren,[7] developing an analytical procedure for transfering moment between columns and a flat plate floor, suggested that the lateral force moments be split into two parts; one part was carried in direct bending from the slab into the column. The critical section was assumed at a distance equal to the slab thickness outside the column periphery. The other part, in the same critical section, was carried by torsional moment. Concrete and steel bars passing through this section were considered to produce resisting shearing stresses.

The studies cited above investigated the interaction of flat plates and laterally loaded columns or rectangular shear walls. In this study, an attempt is made to corelate with results of previous studies and to extend them to include shear walls of various sizes and shapes.

CHAPTER II

THEORETICAL CONSIDERATIONS

2.1 Introduction. The general elastic plate theory has been used by Carpenter[4] to analyse a flat plate to which couples have been applied by laterally loaded columns. While finite element or finite difference methods could be used to extend this analysis to plates loaded through shear walls, only an experimental study is undertaken here. Plate deflections and slopes under known loadings are measured and the plate stiffness and carry-over factors calculated from the appropriate applied loads and measured deflections.

2.2 Stiffness Factors. The definitions of the stiffness and carry-over factors used for the plate are those normally used in the slope - deflection equations. The factors are derived by considering a member AB subjected to transverse loads and end forces as shown in Fig. 2.2.1

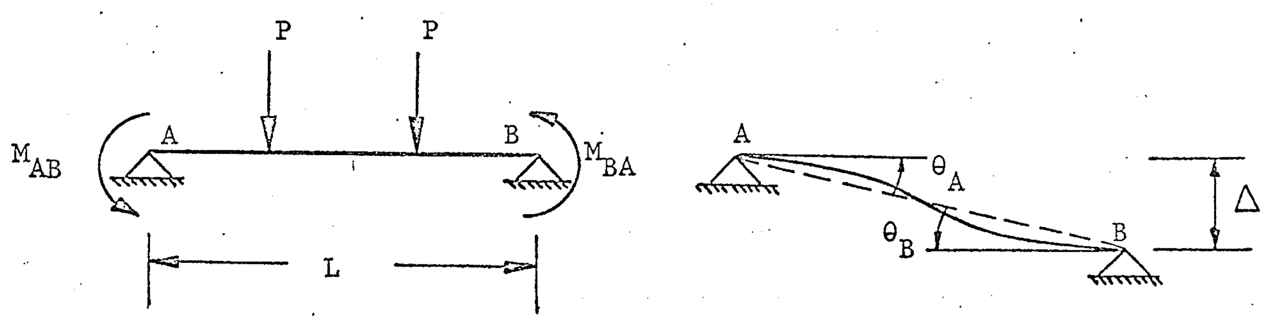

Fig 2.2.1 Loads and Displacements for Member AB

Assuming clockwise moments and rotations to be positive, and assuming displacement to be positive for clockwise rotation of the member about end A (all signs shown here are negative) the following relationships are obtained:

$$M_{AB} = \frac{2\,E\,I}{L}\left(2\theta_A + \theta_B - \frac{3\,\Delta}{L} \right) \pm M_{FAB} \qquad \ldots\ldots \quad 2.2.1$$

$$M_{BA} = \frac{2\,E\,I}{L}\left(2\theta_B + \theta_A - \frac{3\,\Delta}{L} \right) \pm M_{FBA} \qquad \ldots\ldots \quad 2.2.2$$

where, M_{AB} , M_{BA} = moments on ends A and B respectively,

M_{FAB}, M_{FBA} = fixed end moments on ends A and B respectively,

θ_A , θ_B = rotations of ends A and B relative to line AB,

Δ = transverse displacement of end B relative to end A,

L = span length of member,

P = any arbitrary load,

E = Young's Modulus of Elasticity,

and I = moment of inertia of the member.

The above two equations are known as the slope - diflection equations. For a continuous flat plate as shown in Fig. 2.2.2, if a moment, M_{AB}, is applied at the support AA, the slope - deflection equation may be simplified as follow:

Let the span length L, the width b and the depth d of the member be the same as the span length L_x, the equivalent width L_y and the thick-

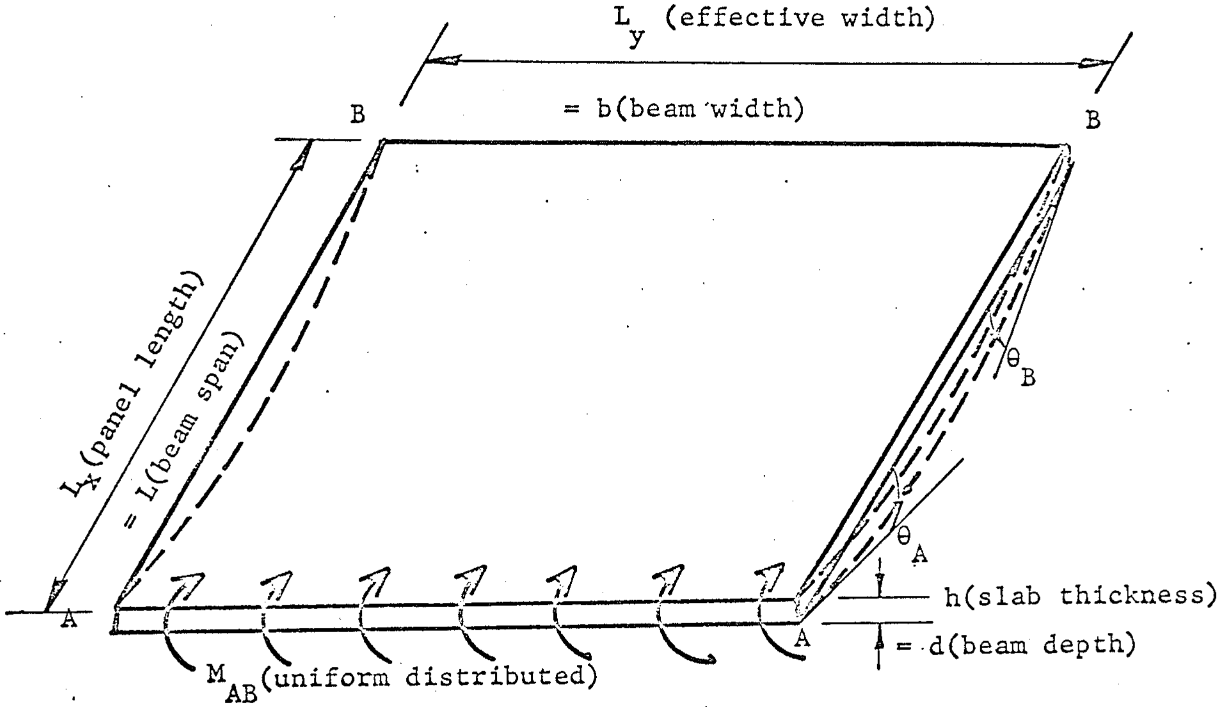

Fig. 2.2.2 Imaginary Beam of a Flat Plate.

ness h of the flat plate, thus creating a hypothetical wide shallow member.

From equation 2.2.1, since $M_{FAB} = 0$,

$$M_{AB} = \frac{2EI}{L}\left(2\theta_A - \theta_B - \frac{3\Delta}{L}\right) \quad \ldots\ldots \quad 2.2.3$$

where θ_A , θ_B = clockwise rotation and counter clockwise rotation of ends A and B of the plate respectively,

I = the moment of inertia of the plate,

and the other terms are defined as for the beam.

If end B of the member is fixed and end A is restrained from translation (pinned), θ_B and Δ become zero and then

$$M_{AB} = \frac{4EI\theta_A}{L_x} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 2.2.4$$

or

$$\frac{M_{AB}}{\theta_A} = \frac{4EI}{L_x} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 2.2.5$$

Equation 2.2.5 gives the moment - rotation relationship for the plate, and is used to define the stiffness factor which is designated as

$$K = \frac{M_{AB}}{\theta_A} = \frac{4EI}{L_x} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 2.2.6$$

where

$$I = \frac{L_y h^3}{12} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 2.2.7$$

Combining equations 2.2.6 and 2.2.7

$$K = \frac{4E}{L_x}\left(\frac{L_y h^3}{12}\right) = \frac{E L_y h^3}{3 L_x} \qquad \ldots\ldots \quad 2.2.8.$$

The stiffness factor can be determined experimentally by applying known moments M_{AB} and measuring the resulting plate rotations θ_A. When the stiffness factor has been determined, the effective width of the plate, L_y, can be calculated from equation 2.2.8, and is:

$$L_y = \frac{3 K L_x}{E h^3} \qquad \ldots\ldots \quad 2.2.9$$

2.3 <u>Carry - Over Factors</u>. The carry-over factor for a member AB is the ratio of the moment M_{BA} produced at fixed support B, to the moment M_{AB} applied at a simply supported end A. Since it is difficult to measure the fixing moment at a plate support, the principle of superposition was used in obtaining the carry-over factors.

To illustrate the procedure beam ABC in Fig. 2.3.1 is considered. End C is fixed while points A and B are simply supported. In Fig. 2.3.1(a), a clockwise moment, M_{BA}, is applied at B and caused rotations θ_{A1} and θ_{B1} at A and B respectively. Moment, M_{BA}, is now removed and a clockwise moment, M_{AB}, is applied at end A as shown in Fig.2.3.1(b). The resulting rotations at A and B are θ_{A2} and θ_{B2} respectively.

Since M_{BA} causes a rotation θ_{A1} at A, and

M_{AB} causes a rotation θ_{A2} at the same point A, the moment at A required to produce rotation θ_{A1} at point A is $M_{AB}\ \theta_{A1}\ /\ \theta_{A2}$.

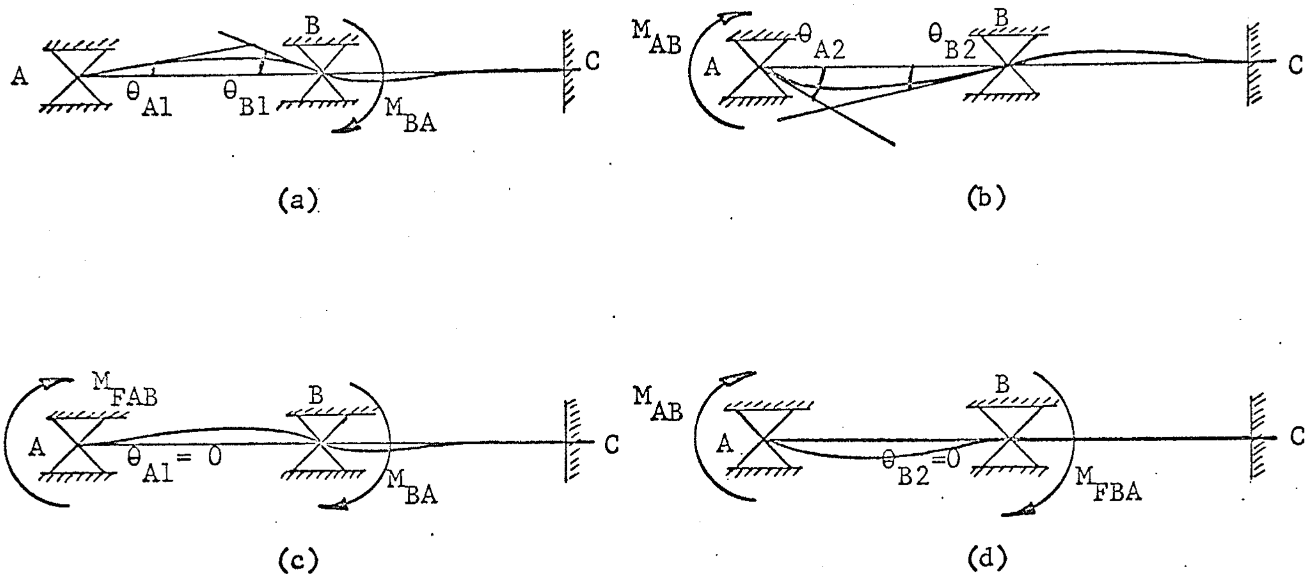

Fig. 2.3.1 Superposition

If the total rotation at A is to be made zero, an imaginary clockwise moment equal to $M_{AB}\ \theta_{A1}\ /\ \theta_{A2}$ should be applied at point A at the same time as M_{BA} is applied at point B. This imaginary moment is called the fixed end moment A to B or M_{FAB} as shown in Fig. 2.3.1(c).

Then the carry-over factor due to M_{BA} is equal to the fixed end moment M_{FAB} divided by the moment $M_{BA}/2$ or

$$C_{BA} = \frac{2\ M_{AB}\ \theta_{A1}}{\theta_{A2}\ M_{BA}} \qquad \ldots\ldots\ \ 2.3.1$$

In the same way if θ_{B2} is to be kept at zero when M_{AB} is applied, the fixed end moment B to A or M_{FBA} is equal to $M_{BA}\ \theta_{B2}/\ \theta_{B1}$ as shown in Fig. 2.3.1(d). Therefore,

$$C_{AB} = \frac{M_{BA}\ \theta_{B2}}{\theta_{B1}\ M_{AB}} \qquad \text{.............. } 2.3.2$$

Equation 2.3.1 is used to determine the carry-over factors for panels loaded at interior columns of shear walls, while equation 2.3.2 is used to determine the carry-over factors for panels loaded at exterior columns.

2.4 Measurement of Stiffness Factor for Model C1. For micro - concrete model C1 (shown in Fig. 2.4.1), the section modulus of the loaded column was very small compared to that for the plate and it was impossible to obtain sufficiently large plate deflections by applying lateral loads to the column. Consequently, the following procedure had to be used to determine the stiffness factor for the plate.

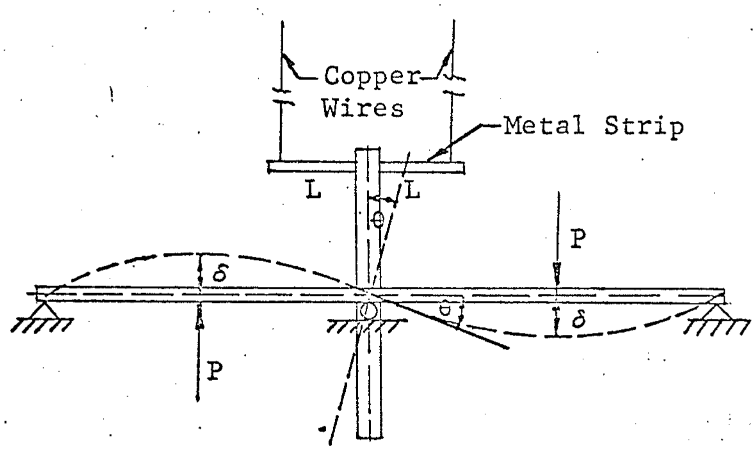

Fig. 2.4.1 Model C1

A pair of loads P were applied to the slab at equal distances from the exterior column as shown in Fig. 2.4.1. A metal strip fixed to the column, was connected to deflection gauges by means of copper wires and used to accurately measure the column rotations.

From the reciprocal theorem

$$2 P \delta = M \theta \qquad \text{............} \quad 2.4.1$$

where θ = the column rotation produced by loads P,

δ = the slab deflection at each load P that would result from a moment M applied to the column.

If M is assumed to be unity, then equation 2.4.1 becomes

$$\delta = \frac{\theta}{2 P} \qquad \text{............} \quad 2.4.2$$

By varying the points of load application, a series of θ values was obtained and the corresponding deflections calculated and plotted. The slope α of the plate was then measured from the deflection curve, and the stiffness factor calculated from;

$$K = \frac{M}{\alpha} = \frac{1}{\alpha} \qquad \text{............} \quad 2.4.3$$

stiffness calculations for loads P of 50, 100 and 150 lbs. are included in Appendix C.

CHAPTER III

EXPERIMENTAL PROCEDURE

PART A PLASTIC MODELS

3.1 Description of Models. Four 1/24 scale flat plate models designated P1, P2, P3 and P4 were tested. Their dimentions are given in Figs. 3.1.1, 3.1.2, 3.1.3 and 3.1.4 respectively. Panel widths for models P2, P3 and P4 were varied to accomodate shear walls of various shapes.

The models were designed in such a way that loading could be applied through either the center column or shear wall, or one or two of the exterior columns along the same loading line. All exterior columns remained fixed when the center column or a shear wall was being loaded.

The models were cut from a sheet of 1/4 in. thick black plexiglas (acrylic sheet). Fixed columns were simulated by column stubs cut from 1/2 in. square steel bar with 1/4 in. diameter holes drilled through their centres to clamp them to the plate and loading frame. They were 1/2 in. in height except for those in the line of loading, which were 3/4 in. in height. The loaded columns had the same cross-sectional dimensions as the fixed columns and were 3½ in. in height. A V-notch was made on each loaded column stub at 2 7/8 in. from the plate in order to provide a 6 in. moment arm. The rectangular shear walls were cut from black plexiglas sheets and cemented to the top and bottom of the plate to simulated a shear wall passing through a flat

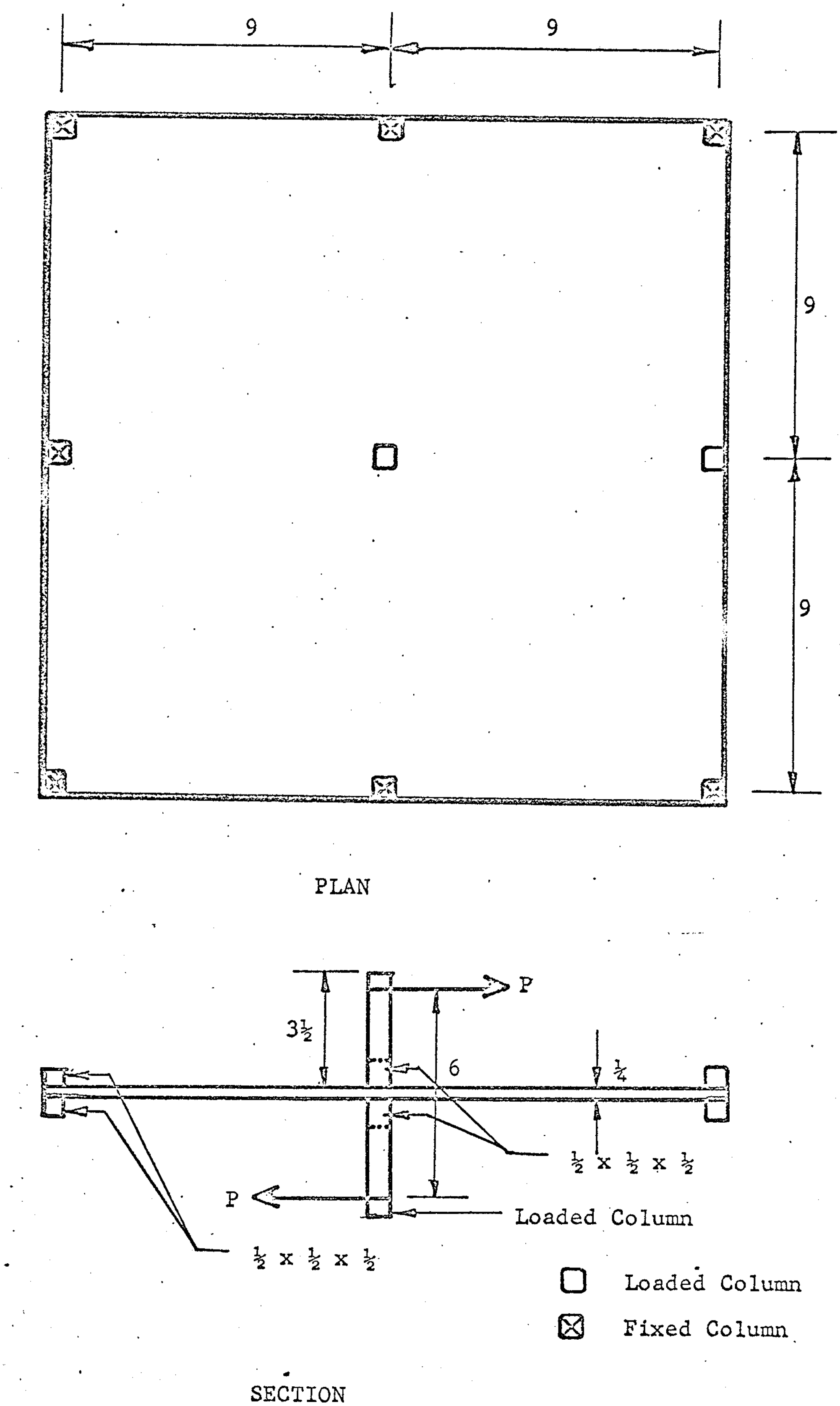

Fig. 3.1.1 Model P1