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A MODEL ANALYSIS OF FLAT SLABS

by

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ABSTRACT

A two-storey 3 bay x 3 bay, 1/12 scale, perspex flat slab model was tested under a one-bay uniformly distributed load, which was applied successively to the various bays. The load was applied using a rubber air bag. The moments induced in the model were measured by electrical resistance type strain gages and the deflections by dial gages.

By analysis of the experimental results, and comparing them with the ACI 318-71 Direct Design Method, the following conclusions were obtained:

The total static moment given by the ACI 318-71 Direct Design Method is accurate for all practical purposes. The width of the column strip specified by the ACI Code in Clause 13.1.2 of 0.25 ℓ_2 but not greater than 0.25 ℓ_1 results in too great a concentration of negative moment in the column strip. This concentration of moment in the column strip is even less justified for the positive moment where the column strip moment/unit width and the middle strip moments/unit width are of approximately equal magnitude. The effect of the concentration of positive moment in the column strip together with aforementioned restriction of the width of the column strip results in middle strip design positive moments 25% less than those measured. The provision "13.3.4.6" that a design moment may be adjusted by up to 10% provided that the total static moment is not less than required should be changed by eliminating the 10% limit. It is recommended that adjustment be applied to the negative moment only. The provisions for the effects of pattern loading "13.3.6" are insufficient. The limitation of live load to dead load ratios stated in "13.3.1.5" results in a possibility of developing negative moments in or near mid span of interior panels.

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CHAPTER 1

INTRODUCTION

Reinforced concrete flat slabs are a near ideal structural element from the viewpoints of formwork design and fabrication, ease and speed of construction, flexibility of use of space enclosed, aesthetics and fire protections. Because of these inherent advantages reinforced concrete flat slabs are the preferred type of design for low to medium rise apartment buildings, office buildings and warehouses.

A flat slab is basically a multi-bay reinforced concrete slab carried directly by columns. However, the term flat slab can also be taken to include slabs which are thickened in the vicinity of the columns by either column capitals and/or drop panels. A close relative of the flat slab is a slab with beams along the column lines. Waffle slabs are simply flat slabs with concrete omitted from some regions where the concrete would be in tension.

The reinforced concrete flat slab was first used by Turner in 1906 (72)[*] for the C. A. Bovey Building in Minneapolis.

The popularity of this "slab without beams" increased rapidly over the years such that over a 1000 examples are known to have been built by 1913.

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[*] All numbers in parenthesis refer to a reference number in the List of References.

The advent of the flat slab marked a transition from planar structures composed of two dimensional elements to monolithic structures composed of elements whose behaviour was mostly more difficult to understand. The design of early flat slabs was largely intuitive and empirical in nature and relied, successfully, on the understanding of the behaviour of this form of construction by their builders. The first comprehensive study on flat slabs was published by Westergaard and Slater in 1921 (78).

In 1920 the ACI (American Concrete Institute) Committee on Reinforced Concrete and Building Laws (9) attempted to provide guidance on the design of flat slabs by providing a design method. Since the development of the flat slab was practical in nature the design equations provided to obtain design moments were empirical and even violated the principles of statics. This was due to intuitive use, perhaps understanding, of the concepts of moment redistribution, the necessity to develop a mechanism for collapse to occur, and the probability of all bays or floors being loaded simultaneously.

Even though modifications were made to the code formulae over the years, as recently as 1963 the Building Code Requirements of the ACI (ACI 318-63) contained provisions for the design of flat slabs that were empirical in nature.

In the years 1956 through 1969 Sozen, the PCA and others engaged in a major research program into the behaviour of flat slab type structures. This work, together with the improved understanding of the behaviour of structures resulted in a complete revision of the code provision of the design of flat slabs in the 1971 version of the Building Code Requirements of the ACI (ACI 318-71).

ACI 318-71 describes two procedures for the design of 'multiple slab systems', namely the Direct Design Method (DDM) which is restricted to slabs which satisfy certain criteria, and the Equivalent Frame Method (EFM).

1.1 OBJECT AND SCOPE OF INVESTIGATION

The basic object of this investigation was to determine the validity of some of the clauses used by the ACI 318-71, for the design of slab systems with multiple square or rectangular panels, namely: 13.1.2, 13.1.3, 13.3.2.1 through 13.3.4.3, 13.3.4.5, 13.3.4.6 and 13.3.6.

A further clause which it was thought might cause difficulties with the Direct Design Method was 13.3.1.5.

This investigation consisted of an indirect model test of a two storey 3 bay x 3 bay perspex flat plate model under a one bay uniformly distributed load, for each storey, which was applied successively to the various bays. The moments induced in the model were measured by electric resistance type strain gauges and the deflections by dial gauges.

PART I. THEORY

CHAPTER 2

AN OUTLINE OF THE FLAT SLAB PROBLEM

2.1 TYPES OF REINFORCED CONCRETE SLABS

Slabs are classified by the way they are supported. Those slabs which are supported such that they essentially carry load in only one direction are known as one-way slabs. Those slabs which are supported by supports which permit the slab to carry the load in two orthogonal directions are classified as two-way slabs.*

1. One-way slabs

The structural action of the slab is essentially one-way, and the slab can be treated as a rectangular beam of comparatively large ratio of width to depth, this large ratio results in shear stresses that are less important in one-way slabs than in beams. Transverse steel is needed to care for shrinkage stresses, temperature stresses and to assist in distributing concentrated loads. Some of the most common examples of the one-way slabs are shown in Figure 2.1.

2. Two-way slabs

The structural action of the slab is in two directions. The usual two-way slab is square or rectangular with the ratio of long span to short span less than two. To improve performance and economy,

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*Up to the current ACI Building Code requirements (ACI 318-71) a "two-way" was used to designate a slab system with beams in both directions (72).

two-way slabs may be stiffened by the addition of beams between columns, thickening of the slab in the vicinity of the column (drop panels), or by the flaring of the column cross section directly under the slab (column capital).

Figure 2.2d shows an example of a slab with beams (formerly two-way slab). The slab may rest on reinforced concrete or steel beams or on walls.

Figure 2.2a shows an example of a slab without beams (formerly flat slab). The slab is supported directly on the column which may have capitals and the slab may have drop panels, the capital can be hidden in the columns and the drop panels in the slab by increasing cross-sectional dimensions of the column to include the capital and making the slab thicker to include the drop panels. The resulting structure is another type of slab without beams (formerly flat plate) (Figure 2.2b,c).

3. Irregular slabs

The supports of the slab are random in size and location and the slab systems may exhibit both one-way and two-way behavior.

2.2 ADVANTAGES OF FLAT SLAB FLOORS

Since formwork represents over half the cost of reinforced concrete slabs, economy of formwork often means over-all economy.

For heavy live loads, that is, over 100 psf, flat slabs have long been recognized as the most economical type of construction (67).

Due to lack of need for beams the storey height can be reduced, the ceiling is smooth, and there is more flexibility of adjusting column locations to fit the room arrangements. Owing to the lack of sharp corners, the flat slab floor is better able to resist continued exposure to fire than the beam and girder floor. (It has been found by experience that the worst damage caused to reinforced concrete by severe fires has occurred at places where there may be spalling, i.e., at exposed edges and sharp corners).

Automatic-sprinkler protection may be made more complete under a flat slab floor, since the nozzles may be placed well up near the under side of the slab without obstruction to the path of the spray.

More light may be admitted into the building if desired, by placing the wall beams above the floor level and thus allowing the windows to be extended to under side of the slab. The absence of deep beams and girders also removes the obstruction to the passage of light within the building.

During construction the opportunity for inspecting the position of the reinforcement is excellent, and the conditions attending deposition and placing of the concrete are favorable to receiving uniformity and soundness in the concrete.

2.3 HISTORICAL DEVELOPMENT OF FLAT SLAB DESIGN

When studying the historical development of flat slab design it can be divided into two main areas:

1. Development of working-stress design methods and theories.
2. Development of ultimate-strength design methods and theories.

Although the two areas overlap, it is worthwhile to treat them independently, rather than present them as one.

2.3.1 WORKING-STRESS DESIGN

In 1903, Turner received a patent for a "mushroom" slab, that is, a slab supported directly by columns with no beams spanning between columns. In 1906 Turner built the first flat slab in Minneapolis but, since there was no rigorous mathematical basis for the design, a performance load test was required (78). While the early proof tests were not adequate to form a basis for design recommendations, suffice it to state that, using load tests, Turner's flat slab and the similar slabs of many of his competitors were deemed satisfactory. For almost 15 years following its introduction, the flat slab was the subject of considerable controversy amongst the engineering profession. A comparison of the differences of opinion on the design of flat slabs was illustrated by McMillan in 1910 (25,63).

In this comparison (Figure 2.3) McMillan indicated the amount of reinforcement required by various design procedures in a 20 x 20 ft interior panel of an 8 in. thick flat slab, intended to carry 200 lb per sq ft. It can be seen that the weight of steel required could vary by 400 percent depending on the design method chosen.

In 1910, Lord made one of the earliest investigations into the behavior of flat slabs by testing a full scale building slab (The Deere and Webber Building) in Minneapolis, Minnesota (62). Unfortunately, misinterpretation of the test data did little toward resolving the mystery of flat slabs.

By 1913, over 1000 flat slab buildings had been built around the world.

In 1914 Nichols presented a simple straightforward analysis of flat slabs (65). He considered the equilibrium conditions of one rectangular panel of a plate having an infinite number of identical uniformly loaded panels and suggested that the absolute sum of positive (M_A) and negative (M_B) moments should add to the total static moment M_o. Thus,

$$M_A + M_B = M_o = \frac{WL}{8} \qquad (2\text{-}1)$$

where W was the total load on the panel and L was the length of span in the direction moments are being determined.

Equation (2-1) does not indicate the relative magnitude of the positive and negative moments. For an interior panel of a flat slab with a round column (or column capital) and with the assumption that the reaction is distributed uniformly around the column capital (ignoring the twisting moments between the slab and the capital) Equation (2-1) becomes

$$M_o = \frac{WL}{8}\left[1 - \frac{4c}{\pi L} + \frac{1}{3}\left(\frac{c}{L}\right)^3\right] \qquad (2\text{-}2)$$

where c was the size of rectangular or equivalent rectangular column, capital or bracket, measured in the direction in which moments are being determined.

Nichols then suggested that the above equation could be approximated to:

$$M_o = \frac{WL}{8}\left(1 - \frac{2c}{3L}\right)^2 \qquad (2\text{-}3)$$

Nichol's analysis was not accepted since the theoretical findings did not agree with practical experience and (misinterpreted) test results (74).

This lack of agreement added to the existing confusion. Journals became a battleground of varying opinions, with very little addition to engineering knowledge. Many theories were put forward to explain differences between Nichol's analysis and the experimental results. Some of these theories were entirely erroneous, while others contained only a portion of the truth.

Most of the reasons for this lack of agreement are now recognized. The major ones are:

a) Some of the moment was carried by the concrete in tension.

b) Twisting moments in the vicinity of the column were neglected.

c) Arch action existed, which increased the compressive stresses and decreased the tensile stresses in a flat slab.

d) The average steel strain over the gage length was measured and this is less than the strain in the steel for the section through a crack, and

e) The effect of adjacent unloaded panels was ignored in estimating the moment capacity and the factor of safety of a slab. (For example, the factor of safety for slab J tested at Purdue University was based on a capacity of 872 lb per sq ft which was obtained when only one of the four panels was fully loaded).

The first joint committee (ACI-ASCE) on concrete and reinforced concrete adopted the form but not the intent of equation (2-3). In the final committee report published in 1916 (55), the design static moment was given as 85 percent of that based on Nichol's analysis. The 1917 proposed revision (8) and the 1920 approved version (9) of the ACI Building Code contained another form of equation (2-3) that was described by Sozen (72) as "an even more flagrant violation of statics". The equation criticized was

$$M_{od} = 0.09\ wL_2(L-qc)^2 \tag{2-4}$$

where qc was twice the distance from center line of the capital to the center of gravity of the periphery of the half capital.

In 1921, Westergaard and Slater published the first comprehensive study on flat slabs (78). They presented solutions of the elastic plate equations relevant to flat slabs, and a comprehensive study of the implications of the then available tests on flat slabs and two-way slabs. Results of the analysis were shown in a comprehensive series of diagrams indicating the distribution of moments at various sections, and tables indicating the percentage of the total moment which was resisted in the critical design sections. These percentages formed the basis of the empirical design method (3).

In 1921, a slightly modified version of equation (2-4) was considered by Slater, namely designing for a total moment of

$$M_{od} = 0.09\ WL(1 - \frac{2c}{3L})^2 \tag{2-5}$$

This moment value was adopted by the Second Joint Committee on Specifications for Reinforced Concrete in 1925 (56) with a footnote, "The sum of the negative and positive moments provided for by this equation is about 72 percent of the moment found by rigid* analysis based upon the principles of mechanics. Extensive tests and experience with existing structures have shown that the requirements here will give adequate strength."

The 1928 ACI Building Code (7) provided equation (2-5) without the footnote.

The empirical method had some restrictions in its applications: Slabs had to consist of a minimum of three continuous spans in each direction, the panels had to be nearly square and span ratios were only allowed to vary over a small range. Designers needed a method with greater freedom in its applications.

In 1941 design by elastic frame analysis made its first appearance in the "ACI 318-41" Building Code (1). It is a two-dimensional approximation to the plate problem which considers the structure as a series of bents, each consisting of a row of columns or supports, and strips of supported slab one panel in width. Bents along column lines in both directions are analyzed for the full load on the floor area. As equilibrium was satisfied, the sum of the calculated moments for a panel was the full static moment rather than 72 percent of the static moment as specified in the empirical method.

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*Should be "rigorous".

However, it was modified to yield answers comparable to those of the "empirical method." A reduction was made by stipulating the design negative moment to be that at a distance xL, from the center of the column width, where

$$x = 0.073 + 0.57\,\frac{A'}{L} \tag{2-6}$$

where A' was half the capital width but not greater than L/8.

The 1956 ACI Building Code, (ACI 318-56, (2) introduced a new factor "F", into the design by "empirical method", based on the fear that the old tests on flat slabs which had c/L ratios of the order of 0.2 might not apply to the modern flat plate which had rather low c/L ratios. Thus equation (2-5) was changed to:

$$M_{od} = 0.09\ WLF\left(1 - \frac{2c}{3L}\right)^2 \tag{2-7}$$

where $F = 1.15 - \frac{c}{L}$ but not less than 1. (2-8)

Also, in the 1956 ACI Building Code, ACI 318-56, further modifications were made to the ACI elastic frame analysis. The moment reduction was further disguised by dropping equation (2-6) and specifying a physical definition of the critical section for negative moment. It was also permitted to reduce the design moments in such proportion that the numerical sum of the positive and average negative bending moments used in design procedure need not exceed M_o as given by equation (2-7), if the structure analyzed was within the range of the empirical method.

The 1963 ACI Building Code, ACI 318-63, did not introduce any major changes in the design of flat slabs.

In 1956, an extensive testing programme of multiple-panel reinforced concrete floor slabs was started at the University of Illinois (12,36,37,42,43,44,52,53,72,76,77,82). Tests were conducted on models of full scale prototypeestructures, 1/4 scale models of slabs with 20' x 20' panels arranged in 9 bays, 3x3, two of the models were flat plates, and two flat slabs, designed in accordance with the provisions of ACI 318-56 (2). In conjunction with these tests a 3/4 full-scale model of the flat plate studied in the Illinois programme was tested in the Portland Cement Association Laboratories(40). The main aspects of flat slab behavior that were studied, with the objectives of re-evaluating the knowledge accumulated from previous work and to prepare advanced design methods for multiple-panel floor slab systems. Among the criteria considered in this investigation were: deflections, bending moments, crack patterns, modes of failure and loads at failure.

The conclusions drawn from these tests and analytical studies (31,32) resulted in drastically revised provisions for the design of multi-panel slab systems in ACI 318-71 Building Code. Those conclusions will be reviewed in the following chapter of this investigation.

A comprehensive study of flat plate structures began in Australia in 1959 by the Division of Building Research at the CSIRO (14 to 18,22,23,38,59). Two reinforced concrete and two prestressed concrete flat plates were tested. The CSIRO's first test slab represented an extreme case for long term deflection behavior. The second test slab had all the negative reinforcement uniformly distributed over the column strip. The two prestressed concrete flat plates showed superior load-deflection characteristics.

2.3.2 ULTIMATE-STRENGTH DESIGN

Since the ultimate strength of flat slabs is beyond the scope of this thesis, this stage of development is not considered in detail.

Historically, ultimate strength was the earliest method used in design since the ultimate load could be measured by test without a knowledge of the magnitude or distribution of internal stresses.

Although the study of the flexural behavior of plates up to the ultimate load* may date back to the 1920's (49) the fundamental concept of the yield line theory for the ultimate load design of reinforced concrete slabs has been expanded considerably in 1943 by Johansen (54) and others (47,57,81). Collapse mechanisms were examined and upper bound solutions to the ultimate load carrying capacity was determined for a preselected layout of reinforcement. Yield line methods give upper bound solutions in theory but due to the idealization assumptions used to obtain the plastic moment, experimental investigations generally show the theoretical values to be conservative.

In 1956 Hillerborg published a paper (45) proposing a simple design method for reinforced concrete slabs. Since lower-bound solutions are valid with variable reinforcement, Hillerborg decided to

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*Not to be confused with ultimate-strength design as used by ACI 318-63, 318-71 which referes to detailing of sections.

invert the yield line procedure for design purposes and make the reinforcement exactly fit the stress field. His theory states that at failure no load is carried by the twisting strength of the slab but is carried either by bending in the x direction or by separate bending in the y direction (x and y are two orthogonal directions).

In 1956 ACI Code, ACI 318-56, (2) officially recognized and permitted the ultimate strength method of design for the detailing of sections or members.

The 1963 ACI Code, ACI 318-63, (3) treated the working stress method and the ultimate strength method on an equal basis for section detailing purposes.

The 1971 ACI Code (4) has entirely accepted the ultimate strength method and relegated the working stress method to a small section referred to as the "alternate method". The term "strength" instead of the term "ultimate strength" has been adopted in the 1971 ACI Code.

CHAPTER 3

THEORETICAL INVESTIGATION

3.1 INTRODUCTION

The design of reinforced concrete structures can be performed by either of two alternative approaches. The first directs attention to stress conditions within the structural member under working loads and is known as working-stress design. The second focuses on the strength capacity of the member at conditions corresponding to failure and is known as ultimate-strength design.

3.1.1 WORKING-STRESS DESIGN

This method (referred to by the 1971 ACI Code as the "alternate method") directs attention to stress conditions within the structural member under working loads. It gives no information regarding the actual factors of safety against failure of a member. The allowable stresses, in this method, must provide a factor of safety against attainment of some upper limiting stress. The stresses computed under the action of service loads will be well within the elastic range, so that the straight-line variation between stress and strain is used. The method assumes that if a factor of safety is achieved against attaining a limiting stress for each component material in the element then the element will have an adequate margin of safety against failure.

3.1.2 ULTIMATE STRENGTH DESIGN

The actual behaviour of a statically indeterminate structure is such that after the ultimate moment capacities at one or more points have been reached, discontinuities in the elastic curve at these points develop and the results of an elastic analysis are no longer valid. There would be redistribution of bending moments throughout the structure until a sufficient number of sections of discontinuity, commonly called "plastic hinges", form to change the structure into a mechanism, at which time the structure collapses or fails. The structure or structural element is then designed to provide the desired ultimate strength. By this approach the method takes into account the nonlinear stress-strain behaviour of concrete, and permits an evaluation of the actual factor of safety against failure. However, it gives little information about conditions at service loads.

3.1.3 COMMENTS AND CONCLUSION

No matter which of the two design methods is employed, the designer must consider both strength and serviceability of any structural element.

Serviceability factors that may be of great importance are unsatisfactory deflection, detrimental cracking, excessive amplitude or frequency of vibration, and excessive noise transmission.

It is quite possible that in the near future the "alternate method" will disappear and the only use for the elastic concepts of the working stresses method will be in the computation of deflections, which are of interest under service loads rather than at ultimate strength.

3.2 METHODS OF SLAB ANALYSIS

An exact elastic analysis, for reinforced concrete flat slabs, requires the use of numerical techniques such as finite difference (11) or finite element (29,35) procedures. These methods are not suitable for routine design use because of their complexity and length. Fortunately several simplified procedures for the design of slabs have been developed.

The reliable ways for the analysis of slabs, at present, are:

1. Codes of practice methods (the methods given in ACI 318-71 only will be discussed here).
2. Elastic analysis methods.
3. The yield line method.
4. The strip method.
5. Optimum placing of reinforcement by computer analysis, based on elastic stress fields coupled to the orthotropic yield criterion.

3.2.1 ACI 318-71 DESIGN METHODS FOR TWO-WAY SLABS

3.2.1.1 Introduction

In 1956 an extensive experimental program and analytical studies on reinforced concrete slabs was started at the University of Illinois (12,36,37,42,43,44,52,53,72,76,77,82) in coordination with another experimental program at the Portland Cement Association laboratories (40). These experimental and analytical studies were conducted for the purpose of providing a basis for more rational design methods, for reinforced concrete slabs, than those which were in use (2,3). The experimental program at the University of Illinois involved testing of one-quarter scale models of various floor systems. To aid interpretation of the one-quarter scale model tests, a flat plate structure constructed at three-quarter scale and 45 ft. square was tested at the PCA laboratories.

Some of the significant conclusions drawn from the testing program and of analytical studies (31,32) were:

1. The sum of bending moments calculated from measured steel strains agreed reasonably well with the total static moment calculations.
2. The mode of failure of flat slabs with column capitals and drop panels is generally flexural.
3. Failure of flat plates will usually occur, due to punching shear, before full flexural capacity is reached.
4. Empirical equations for shear strength, derived by Moe (64), may overestimate the shear capacity of flat plates.
5. At working loads, deflections and steel stresses are small and cracking is usually restricted to the column head region, which may justify a greater proportion of the slab steel being sited in the column head region than the proportion obtained from a reinforcement distribution based on elastic analysis.
6. Yield-line theory underestimates the failure load but usually

predicts the critical yield pattern. Crack patterns foreshadow the likely failure mode.

7. The fact that the flexural capacity of the slab is greater than that predicted by yield-line theory, indicates that compressive membrane forces may be present.
8. For the same loading flat plates have higher deflections and stresses than flat slabs with column capitals and/or drop panels. However, for slabs designed by ACI 318-56 Building Code, the flat slab with column capitals and/or drop panels is more highly stressed than the flat plate of the same depth at their respective design loads.

The results of the testing program and of analytical studies resulted in drastically revised provisions for the design of multi-panel slab systems in ACI 318-71.

The design methods for reinforced concrete two-way slabs (flat slabs and flat plates included) that are proposed by the ACI 318-71 contain three major changes (compared with ACI 318-63):

1. a unified approach to the design of all two-way slab systems consisting of reasonably rectangular panels supported on isolated columns with or without beams, drop panels and column capitals, and supporting predominately distributed loading,
2. ultimate strength design for the proportioning of reinforcement in a section has been extended to slabs on an equal basis with other concrete members,

3. the load factors on service loads for the determination of the ultimate design load have been reduced from 1.5 to 1.4 for dead load, and from 1.8 to 1.7 for live load. This reduction in load factors was implemented because of more comprehensive code provisions, additional research and experience and improved concrete and steel quality control.

A copy of, ACI 318-71, Chapter 13 (Slab Systems with Multiple Square or Rectangular Panels) and Commentary on, ACI 318-71, Chapter 13 is given in Appendix D. This copy is revised up to the 1977 Supplement to the Building Code (ACI 318-71).

3.2.1.2 Design Procedure

The ACI 318-71 procedure for the design of two-way slab systems can be summarized in the following steps:

1. Slab thickness: The minimum thickness of slabs or other two-way construction for floors designed in accordance with the provisions of Chapter 13 of ACI 318-71, and having a ratio of long to short span not exceeding two, shall be governed by the following equations, and the other provisions of ACI 318-71, Section 9.5.3

$$h = \frac{h_m}{1 + .139[\alpha_m\beta - 0.5(1-\beta_s)(1+\beta)]}$$

but not less than

$$h = \frac{h_m}{1 + .139\ \beta(1+\beta_s)}$$

the thickness need not be more than h_m where $h_m = \frac{\ell_n(800+.005\ f_y)}{36,000}$ which gives $0.03056\ \ell_n(\ell_n/32.7)$ for $f_y = 60,000$ psi.

The above equations yield thicknesses consistent with those found from experience to provide satisfactory control of deflections for flat slabs, flat plates and conventional two-way slabs supported on beams.

2. Auxiliary elements, if any: the choice of auxiliary elements will depend on such factors as panel location and geometry, and stiffness and shear requirements. These factors are discussed in detail in ACI 318-71, Sections 9.5.3 and 11.10 through 11.13.

3. Design moments at the critical sections: Computation of the slab design moments at the critical positive and negative sections may be performed by either of two methods: (a) The Direct Design Method (see Appendix D, Section 13.3): a simpler method which may be used for slab systems meeting the limitations listed for this method or (b) The Equivalent Frame Method (see Appendix D, Section 13.4). The Equivalent Frame Method must be used when the limitations listed for the direct design method are not satisfied.

4. Proportion slab design moments between column and middle strips: Irrespective of whether the Direct Design Method or the Equivalent Frame Method is used, to determine the total moment on a section, this total moment must then be

proportioned across the slab. The slab is divided into a series of transverse and longitudinal strips parallel to the column lines, each to be proportioned to resist a specified percentage of the negative, or positive, design moment. That portion of the design moment assigned to a particular strip may be assumed to be distributed uniformly across that strip (for more details see Appendix D, Section 13.3.4).

5. Design for shear and torsion: The designer must be aware that the vital problem related to safety of the slab system is the transmission of load from the slab to the columns by flexure, torsion and shear. Design criteria for shear and torsion in slabs are discussed in detail in ACI 318-71, Sections 11.0 through 11.13.

6. Reinforcing: see Appendix D, Section 13.5.

An example of computing the design moments, of a flat slab, at the critical sections, using the Direct Design Method (ACI 318-71) and the proportioning of these moments between the column and middle strips is given in Appendix C.

3.2.2 ELASTIC ANALYSIS OF PLATES

The elastic analysis of reinforced concrete flat slabs has two main facets:

a) The analysis of a "medium-thick" elastic plate.

b) The modifications required because of the characteristics of reinforced concrete.

A discussion of the various elastic analysis methods would be too lengthy and beyond the scope of this thesis. However, a list of those methods with some references is given below:

a) Exact solutions (26,27,75).

b) Finite difference equation solutions (68,78,79).

c) Moment distribution method (13,31).

d) Elastic models (24,60).

e) The finite element method (30,83,84).

3.2.3 YIELD-LINE METHOD

3.2.3.1 Introduction

This method deals with the ultimate load-carrying capacity of slabs, assuming impending collapse. It can be applied to one-way or two-way slabs.

As applied to one-way slabs, it is not much different from the limit analysis of continuous beams as ordinarily treated in plastic design for structural steel.

The chief concern of this theory is the analysis for two-way slabs. Although, not yet adopted by the ACI code, slab analysis by yield line theory may be useful in providing the needed information for understanding the behavior of irregular or single-panel slabs with various boundary conditions.

3.2.3.2 Basic Concepts and Assumptions

Johansen (54) extended the ultimate load analysis of beam and frame structures to reinforced concrete slabs by introducing the concept of yield-lines, which are the two-dimensional counterparts of plastic hinges. The first stage of the yield line analysis of any slab is to predict the yield line pattern at failure (a series of possible patterns must be considered). For a given amount of reinforcement the ultimate resistance moment along the yield lines can be calculated and hence by analyzing the slab at failure the value of the load which is in equilibrium with these moments can be found. Effects other than flexure, such as shear and deflection have to be separately considered. The reinforcing steel is assumed to be fully yielded along the yield lines at collapse and the bending and twisting moments are assumed to be uniform along the yield lines.

Generally yield lines will occur along lines of fixed support, lines of symmetry, and, for slabs supported directly on columns, along an axes passing through the column. Typical yield line patterns of flat-plates are shown in Figures 3.1, 3.2 and 3.3.

Provided that the correct failure pattern is known, the critical load can be obtained either from virtual work or from equilibrium considerations. Both approaches use the following basic assumptions:

1. At impending collapse, yield lines are developed at the location of the maximum moments.

2. The yield lines are straight lines. (This is valid only for distributed loads; point-loads do have curved yield lines).

3. Along the yield lines, constant ultimate moments (m_u) are developed.

4. The elastic deformations within the slab segments are negligible in comparison with the rigid body motions created by the large deformations along the yield lines.

5. Of the many possible collapse mechanisms, only one, pertinent to the lowest failure load, is important. In this case the yield line pattern is optimum.

6. When yield lines are in the optimum position only ultimate bending moments, but no twisting moments or transverse shear forces, are present along the yield lines.

Most books assume in their discussion, that the slab is of uniform thickness and that the reinforcing is the same in both directions (isotropy). The treatment of orthotropic reinforcements, variable thickness, etc., is basically the same. The reader is referred to references (57,58,70,81).

For the analysis of a flat plate three yield line patterns should be considered:

1. One-way collapse modes, either in x or y directions, Figure 3.1a.
2. Simultaneous two-way collapse (Figure 3.1b), in this pattern the critical load remains the same as the previous pattern (Figure 3.1a).

3. Inverted conical collapse mode (Figure 3.1c) centered on each support.

Figure 3.3 shows a collapse mode of a flat plate with hexagonally arranged supports. This mode is really the companion of Figure 2.1b for a square system. It must be noted that for such supports arrangement there are a large number of possible collapse modes.

Figure 3.2 shows a collapse mode of typical interior panels of flat slabs.

3.2.3.3 Summary and Conclusions

Yield-line analysis is a simple and economic limit analysis method, it furnishes realistic upperbounds of collapse loads for any shape, boundary, and loading conditions.

Close agreement has been obtained between experiment and analytical results, in numerous reinforced-concrete independent tests.

Although this method was primarily developed for the design of reinforced concrete slabs, its basic principles are generally applicable for the analysis of all plates, provided that the plate material has the ability to deform plastically and that its stress-strain behavior can be reasonably well approximated by Johansen's rigid-ideally-plastic.

Some limitations to the method are:

- The method gives the total ultimate load-carrying capacity of the plate, this load cannot be applied directly to the design.

- The computation of deflections can be done only via crude approximations.
- Shear must be separately considered.
- Theoretically, the law of superposition is not valid.
- Although, this method can be applied for suddenly applied one-time loads, it fails in vibration analysis and cannot be used in the case of repeated static or dynamic loads.
- The method is not suited to determining a reinforcement layout. Reinforcement layouts determined by elastic analysis give greater control over cracking and deformation at working load.

3.2.4 STRIP METHOD*

In this method the reinforcement is made to fit, exactly, the stress field, which gives the designer a wide freedom of choice in his design approach. Tests of typical slabs designed by this method, and reinforced exactly according to the moments found (without averaging across a band, but with reduction of steel as moment decreased), developed the same load capacity that was introduced into the design. It was concluded, by Armer and Wood, that the design was an almost exact solution rather than just a lower bound solution of the problem** (19,20,80).

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*Developed by Hillerborg.

**The yield-line method is an upper bound procedure.

3.2.4.1 Theoretical Basis

The equilibrium equation for slabs (for elastic plate analysis) is:

$$\frac{\partial^2 Mx}{\partial x^2} + \frac{\partial^2 My}{\partial y^2} - 2\,\frac{\partial^2 Mxy}{\partial x\,\partial y} = -p$$

Hillerborg assumes that the twist moment Mxy = 0, and split up the load p into αp and βp, ($\alpha+\beta=1$), thus the equation can be reduced to:

$$\frac{\partial^2 Mx}{\partial x^2} = -\alpha p$$

and

$$\frac{\partial^2 My}{\partial y^2} = -\beta p$$

while α may be assigned any value, it is usual to consider α as either "0" or "1". Strips are made to carry arbitrary load dispersion diagrams as in Figure 3.4. The designer has to watch the continuity of shear and bending moment in the strips, and make the reinforcement fit.

The flat slab is one of the most difficult slabs for this method. For such a case, Hillerborg developed the advanced strip method (33) in 1959. Hillerborg recommended a rectangular element load in two directions to a support at one corner (the column) of the element. The mathematical basis for this element could not be proved, but a test result, by Armer (20), for a slab design using this method reported satisfactory results. Wood has suggested a wide "beam band" over the columns that could be strengthened at the columns by local (short) strong bands across the column (much like steel shearhead reinforcement in flat plates).

Although, at present the strip method is not ready for everyday design of flat slabs and flat plates, it is believed that development in this particular area are intriguing and appear hopeful.

PART II. EXPERIMENTAL PROGRAM

CHAPTER 4. EXPERIMENTAL INVESTIGATION

CHAPTER 4

EXPERIMENTAL INVESTIGATION

4.1 INTRODUCTION

For practical and economic reasons, it is not often feasible to carry out comprehensive experimental investigations on full size buildings. Consequently the investigation described in this thesis was carried out on a reduced scale perspex model.

The object of the experimental investigation was to measure the bending moments and deflections of a perspex model of a typical two-storey 3 bay by 3 bay flat slab, with rectangular 14" x 16" panels, under various combinations of panels loaded. To achieve the most effective use of the model only one panel was loaded at a time, derived load patterns being achieved by superposition of the results of single panel loaded tests. The model response was measured by surface mounted strain gages and dial gages. The strains were converted into moments by use of the conventional equations. Using those results an investigation was carried out into ACI 318-71, slab systems with multiple square or rectangular panels, clauses: 13.1.2, 13.1.3, 13.3.1.5, 13.3.2.1 through 13.3.4.3, 13.3.4.5, 13.3.4.6 and 13.3.6.

4.2 MODELS IN STRUCTURAL ENGINEERING

Model tests are a long accepted technique in the analysis of redundant structures (34,50,61,66). In discussing models consideration will be confined to small scale three dimensional model studies related to the investigation of reinforced concrete structures.

The models that have been used in studies of reinforced concrete structural elements can be divided into two broad categories, namely direct and indirect models.

4.2.1 DIRECT MODELS

Direct models (10,21,41,71) are assemblies constructed of mortar (micro concrete) reinforced with steel wire. Its properties are identical to those of the proposed design but on a reduced scale.

Direct models must be used if behavior beyond the range of elastic response, or material strength considerations, are important to the study.

The most important advantages of direct models are:

1. Simulation of nonhomogeneity in the prototype.
2. Possibility of observation of behavior over the entire loading range to failure, and
3. Cost as compared to testing of full scale members or assemblies.

The disadvantages of direct models are:

1. The precision required in planning and constructing models.
2. The difficulties in instrumentation of reinforcement.

3. The restriction of a single test to failure as the model is destroyed.
4. The necessity of further laboratory work to prove the inter-relationship, particularly the material properties, between model and prototype, and
5. Certain problems are not answerable to direct model tests without distortion.

4.2.2 INDIRECT MODELS

Indirect models (28,34,50) are constructed of material which is essentially homogeneous and responds elastically to the application of load. Such models, usually built of a synthetic resinous material, are employed to study structural response to load only within the elastic range. Therefore, the results obtained are limited in value to the same extent as the results obtained from an accurate mathematical study founded on the theory of elasticity. Thus, it may be stated that the findings derived from a well conceived elastic model are no better than, but as good as, those from properly executed studies of a mathematical model, except that in experimental models the effect of edge conditions are realized physically rather than assumed on the basis of idealized boundary conditions. The most important advantages of indirect models are:

1. Versatility and ease of constructing simulations of complex configurations.
2. Relative ease in altering configurations as required during the progress of testing.

3. Facility with which strain and distortion measurement may be made, and

4. Simplicity of mechanics of loading as compared to those required for large scale testing.

The disadvantages of indirect models are:

1. Restriction to elastic behavior of model.

2. Dissimilarity of physical properties of the materials, which under certain conditions of strain compatibility, may affect the interrelationship between model and prototype.

All the tests described in this investigation are indirect model tests with the model studies being used to check the accuracy of some clauses of the Direct Design Method of ACI 318-71 (4) for the design of flat slabs.

4.3 THE PROTOTYPE AND THE MODEL

In any investigation into the behavior of structural models the first question to be raised is whether the model is representative of its prototype.

The prototype of the flat slab which has been investigated is illustrated in Figure 4.1. The 6-inch slab is supported directly on 1 ft diameter circular columns which are fixed at their bases. A model of 1/12 scale of the prototype was constructed for this investigation. Scaling of linear dimensions was linear, whereas areas were reduced by the square of the linear scale factor. Loading per square area remained the same, that is 200 psf on the prototype is represented by 200 psf on the model. An initial computation of the stresses in the perspex model* showed that the stresses would be well within the elastic range of the model material (perspex).

*The values of the moments were computed, using 200 psf as the total distributed load on the structure, according to the 1971 ACI Code (ACI 318-71).

4.4 MATERIALS AND CONSTRUCTION OF THE TEST MODEL

4.4.1 CONSTRUCTION CONSIDERATIONS

In very small scale models consideration must be given to the tolerances of dimensions of the structure, the magnitude of the loads and the ability to measure the model's response to the loads. Also ease of construction and access to the model components must be considered.

An error of a small magnitude in the everyday concept of units of measurement may, in the case of the small models, represent a very large relative error in the structure. Choosing ±5 percent as the average allowable construction error, the thickness of the slab should be controlled to within ±0.02 in. This same tolerance can be applied to span lengths and other dimensions.

The necessity of maintaining the construction error as small as possible dictates the time and care that should be spent in the fabrication and construction of the test model. Since the small model means a smaller space to work in, and more cramped places which restrict movements, it was important that the model be designed and oriented in such a way as to provide easy access about, over and under the test model.

4.4.2 TEST FRAME

The frame on which the model was tested consisted of a grid-work of 4-M4 x 13 steel beams, arranged at 16 in. centers. These steel beams were connected by three steel plates, 3" x 1/2" x 54" length, under the bottom side, arranged at 21 in. centers and by four steel plates, 6" x 1/2" x 54" length, on the top side, arranged at 14 in. centers. The bottom plates were rigidly welded to the steel beams, while the top plates were bolt connected, to the top of the steel beams, using 1/2" bolts, to avoid any warping of the top steel plates due to welding heat, and to allow for any necessary adjustments of the distance between the top plates. Half inch holes at 16" centers were provided, in the top plate, to receive the columns of the test model.

Figures 4.2 and 4.7 presents in detail the arrangement of the steel frame on which the model was tested.

4.4.3 DESCRIPTION OF THE TEST MODEL

Perspex was chosen to be the model material as its low modulus of elasticity (0.467 x 10^6 psi), enables a high response (strain) to be obtained for a low load. All the joints were mechanically connected with provisions for possible future changes (column capitals, drop panels, column stiffness). Thickness dimensions were held to a tolerance of plus or minus 0.014 inch on the slab, drop panels, and column capitals and to plus or minus 0.004 inch on columns.

The basic model of the investigation reported herein was a two level flat plate model with no edge beams, drop panels, or column capitals. Each level consisted of nine 14.0 inch by 16.0 inch panels, arranged three by three. The edge strip width was 3.25 inch all around. Columns were 1.0 inch in diameter and 10.5 inch in height. Dimensions of all components of the model are shown in Figures 4.3, 4.4, 4.5, 4.6, 4.8 and 4.9.

The test model was designed with provisions for succeeding future tests, incremental units can be added to form any of the following combinations (see Figure 4.6):

1. The number of levels can be increased up to five levels.
2. Columns stiffness can be changed, by changing column's diameter, height, or both.
3. Flat plate with edge beam.
4. Flat slab with drop panels, column capitals or both.

The columns of the model were supported on a rigid steel base. Preliminary tests indicated no deflection of the supporting structure under loads higher than those encountered in the process of testing.

4.4.4 PROPERTIES OF PERSPEX (48,69)

Perspex, also known as lucite, plexiglass, is a poly-methyl-methacrylate plastic, a water-white solid formed by the polymerization of methyl methacrylate.

The mechanical properties of perspex depend markedly on the temperature at which they are measured, the rate at which the perspex is stressed or strained, and to a less extent on the presence of absorbed water, which tends to act as a plasticiser. This last effect can usually be ignored. A detailed discussion of mechanical properties is given by Hoff (46).

Representative test specimens of the perspex, used in the model, were subjected to simple bending and tensile tests to determine the elastic properties of the perspex.

In the simple bending tests, specimens of section 3.00" x .486" x length 10.00", were simply supported over a span of 8.00" and loaded by a midspan concentrated load. Strains and deflections at midspan were measured using electric resistance type strain gages and dial gages. Stress-strain curves were obtained from the tests and the average modulus of elasticity was 0.467×10^6 psi. The readings of both the deflections and the strains at the center of the specimens helped in checking the gage factor value (2.12).

In the tensile test, specimens of 3" x .486 x 10" length, were subjected to a tensile force acting in the longitudinal direction, strains were measured at the center point in the longitudinal and transverse directions using surface mounted electric resistance type strain gages. Stress-strain curves were obtained from the tests, the average Poisson's ratio was 0.35.

The above values, of the Modulus of Elasticity and Poisson's Ratio were measured at the same temperature at which the model was tested (70° ± 1°F), using the same electric resistance type strain gages and dial gages that were used on the model.

More values for the perspex mechanical properties are listed in Table 4.1.

4.5 INSTRUMENTATION

4.5.1 STRAIN MEASUREMENTS

Provisions were made to measure strains at various sections in the interior and exterior panels so that comparisons to the code (ACI 318-71) results could be made. Strain gages were to be placed in the positive moment regions across the panels centerlines and in the negative moment regions across the column centerlines. The strain gages used were: Kyowa electric resistance strain gages "type KFC-5-C1-11", the gage length of these strain gages was 5 mm, their resistance was 119.8 ± 0.3Ω, and the gage factor was 2.12 ± 1%.

Drawings of the same size as the model's slabs, with the exact locations of the strain gages, were prepared. The drawing was fixed on one side of the slab, and the strain gages were mounted, on their exact location, on the other side prior to the placement of the slab in the model. The strain gages were installed with M-Bond 200 Adhesive.

After the gages had been mounted with all care possible and and a check had been made, on each strain gage, the gages were waterproofed with three layers of M-Coat "A" (air-drying polyurethane coating) at 30 minute intervals.

Several difficulties were encountered during the process of installation, mainly due to the large number of strain gages used (115 strain gages, each had to be welded to three wires). Tricoloured wires were used, and a numbering system of the strain gages and the wires was adopted and executed step by step with the installation procedure of the strain gages. Strain gages were installed in the x-direction on one slab and in the y-direction on the second slab. Tests were done twice for each level using one slab in the first time and the other slab in the second time.

Most of the strain gages were attached to the bottom surfaces of the slabs, though companion gages were attached on the top surface at locations 18, 22, 50 and 54 to determine the presence of axial stresses in the slabs. The locations of the strain gages are shown in Figure 4.12 and Figure 4.13 for gages on the bottom and top surfaces of the slabs.

Some strain gages were mounted on the columns. Their locations are shown in Figures 4.14, 4.15 and 4.16. The measured strains of the columns are given in Appendix A, but they will not be considered in this investigation.

Strains were measured, using a quarter bridge circuit, by a combination of multi-channel digital strain indicator with digital printer module, by a digital strain indicator (Model P-350A) with 1Y.12 switching and balancing unit, and a model SB-1 switch and balance unit.

4.5.2 DEFLECTION MEASUREMENTS

The vertical deflections of the test model were measured at 12 different locations. Deflections at five locations in each panel were measured, thus including all locations at midspan between columns and at mid-panel. Figure 4.17 presents the location and designation of the dial gages. The dial gages used in the test of the test model were 0.001 in. dials with 1-in. stroke. The dial tips were spherical in shape. An individual deflection dial was mounted under each of the points to be considered. Figures 4.8 and 4.9 present typical views of the test model. Figure 4.11 gives a close-up view of the dial gages used to measure the vertical deflections.

4.5.3 LOADING SYSTEM

Individual panel loading was achieved by means of a 16.0 in. by 14.0 in. rubber air pressure bag loading system (39). Several difficulties were encountered in the process of fabricating the airbag. The first airbag was unsuccessful, the second started leaking, through several holes, after a few primary tests. The third was satisfactory, but it started leaking (very slowly) after a few tests, making periodic repairs a necessity. The main problem was with the glue, as the operation necessitated a glue to be very elastic, air tight and strong after drying. Holding the different elements in place while the glue dried was another problem. Different kinds of glues were attempted, and the one which showed best results was "5 Minute-epoxy Lepage". Another airbag has been prepared to replace the one in use in case of any possible problem. The airbag was placed on the top surface of the model and then it was inflated by a hand pump and a system of valves to provide the load. The top reaction for the airbag was provided by a wood plate firmly anchored to the test frame.

The pressure in the airbag was measured by means of a water manometer having a potential error of plus or minus 1/8 in., or 0.65

psf, which was considered to be sufficiently accurate (with intended loading of 200 psf, this error represents plus or minus 0.325%). Details of the airbag and the uniformly distributed loading system are shown in Figures 4.10 and 4.18.

4.6 TESTING OF THE MODEL

Accuracy of strain measurements is the primary consideration in the selection of the magnitude of the test load. Though a high load would produce high strains, the consideration is counterbalanced by desire to use a low load to avoid excessive deformations (second consideration). Accurate measurement of the applied load is the third consideration. A uniformly distributed load of 200 psf (38.5 in. of water) was selected.

Testing was done in an air conditioned building.

The testing of the model began with a test with uniform loading to 50 psf on panel No. P2. Readings of strains and deflections were recorded. The same procedure was repeated with uniform loadings to 100 and 200 psf on the same panel (P2). These tests were merely to check the electrical strain gages, the deflection dial gages, the loading equipment, the test procedure and the linear relationship between the load, the strains and the deflections. Everything worked fine (errors were plus or minus three percent).

The testing of the model was done in two stages: In the first stage the slab with the electrical strain gages in the longitudinal direction was placed in the first level, and the other slab, with the electrical strain gages in the transverse direction, was placed in the second level. In the second stage the two slabs changed levels.

In each stage individual panels were loaded, one at a time, using the uniformly distributed load of 200 psf. This meant 18 individual tests for each stage.

For each test, at the completion of the loading operation, the deflection and strain data was taken. Reading and recording this data required about 25 minutes, which necessitated a very close control of the water manometer by a second person to maintain the pressure in the airbag at 200 psf.

Before each loading the dial gages were set to zero, and the strain gages were individually balanced to zero. The model was then loaded and the strains and deflections measured. The load was removed and strain readings and deflections again taken. The readings due to the load were taken as the difference between readings at load and the readings at the subsequent unloaded condition. After a waiting period of at least three hours the strain gages were balanced again and the dial gages set to zero in preparation for the next test. It can be seen that the strain, and deflection, values are in terms of applied load and do not take into account the uniform dead load on all panels.

Strains in one level due to the loading of a panel of the other level were very small and insignificant.

Table A.1 through Table A.54 present the data derived in the above manner.

PART III. FINDINGS

CHAPTER 5

ANALYSIS OF EXPERIMENTAL RESULTS AND DISCUSSION

5.1 ANALYSIS OF THE EXPERIMENTAL RESULTS

It was noted that the strains of the top and bottom surfaces (measured at locations 18, 22, 50 and 54) closely agree, but are opposite in sign, which indicates a lack of axial or membrane stress in the slab at these locations. This condition prevailed through all tests.

Stresses were calculated from a combination of the measured strains in the two perpendicular directions x and y. Strains and moments were related by pure bending equations (75) as follows:

$$\varepsilon_x = \frac{z}{r_x}$$

$$\varepsilon_y = \frac{z}{r_y} \qquad \text{where } z = \frac{h}{2}$$

$$D = \frac{Eh^3}{12(1-\nu^2)}$$

$$M_x = D\left(\frac{1}{r_x} + \nu\frac{1}{r_y}\right)$$

$$M_y = D\left(\frac{1}{r_y} + \nu\frac{1}{r_x}\right)$$

"ε_x" and "ε_y" denote the strains in the x and y directions, respectively, "$\frac{1}{r_x}$" and "$\frac{1}{r_y}$" denote the curvature of the surface in sections parallel to the xz and yz planes respectively, "D" is the flexural rigidity of the plate, "h" denotes the thickness of the plate, "E" is the modulus of elasticity (0.467 x 10^6 psi for perspex), "ν" denotes Poisson's ration (0.35 for perspex), and M_x and M_y denote the bending moments per unit length acting on the direction parallel to the y and x axis respectively, these moments were considered positive when they produced compression in the upper surface of the plate and tension in the lower.

Substituting the values of E, ν, z and h (0.486 in) in the above equations, the following relationships were obtained:

$$M_x = 20949.176 \quad \varepsilon_x + 7332.212\ \varepsilon_y \quad \text{lb.in.}$$

$$M_y = 20949.176 \quad \varepsilon_y + 7332.2.2\ \varepsilon_x \quad \text{lb.in}$$

(ε_x and ε_y are in inches)

Moments were calculated from the measured strains, using a computer programme (Appendix B) for six different loading patters, (Figure 5.2). Moment values were obtained for a total live load plus dead load equal to 400.0 psf.

Table B.1 and Table B.2 show the values of the strains and moments, for the first and second levels, when all the panels are loaded with a uniformly distributed load of 400.0 psf. Location and orientation of the gages are shown in Figure 4.12.

5.1.1 LOADING PATTERNS

The partial loading patterns for maximum positive and negative moments are shown in Figure 5.1. It can be noticed that there are two main sets of patterns: checkerboard loading patterns and strip loading patterns (51,53).

In this investigation strip loading patterns (Figure 5.2) were used to create maximum loading conditions.

Table B.3 through Table B.16 show the values of the strains and moments, for the different strip loading patterns, when the ratio of the live load to the dead load is equal to 3.64 (L.L. = 313.8 psf and D.L. = 86.2 psf), this value represents a live load to dead load ratio of 3 to 1 multiplied by load factors 1.7 and 1.4 respectively.

The agreement, or otherwise, between the measured moments and the ACI (318-71) moments will be discussed in the following pages.

5.2 COMPARISON OF MODEL TEST RESULTS WITH THE DIRECT DESIGN METHOD (ACI 318-71) AND THE EMPIRICAL METHOD (ACI 318-63)

The moments measured from the flat slab model on this investigation are compared both graphically and in tabular manner with the results calculated using both the Direct Design Method and the Empirical Method*.

Figures 5.3 to 5.6 show the variation of measured and calculated negative moments across the width of the negative moment section for the appropriate loading patterns for an interior panel.

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*It must be noted that bending moments were calculated in each of the two orthogonal directions (x-y) for each of the two slabs of the model. For presentation, the variation of moments can be considered across a section or along a strip.

Figures 5.7 to 5.10 show the variation of positive moment across the slab for an interior panel.

Figures 5.11 to 5.14 show the variation of negative moment across the span for an end span.

Figures 5.15 to 5.18 show the variation of positive moment across the slab for an end span.

Figures 5.19 to 5.22 show the variation of bending moment for end panels.

Tables 5.1 through 5.8 compare the magnitudes and the measured moments with those calculated by the Direct Design Method and the Empirical Method for all panels loaded.

5.2.1 EFFECT OF POISSON'S RATIO

Moments and resulting stresses at a given point in a plate are dependent upon the Poisson's ratio of the material (34,75). Consequently, for two structures of different materials, interpretation of data from one to another should require adjustment to account for differing Poisson's ratios. This has been used occasionally to question the feasibility of model analysis. Because of the complexity of the problem, neither those opposed to nor those favoring models have established sufficient mathematical evidence to prove their viewpoints. However, previous investigations have indicated good agreement between elastic models and reinforced concrete prototypes, even though the effects of differences in Poisson's ratio had been ignored.

Due to the limitations of present mathematical analyses of flat plates, the unknown value of Poisson's ratio of reinforced concrete , the inherent variations in measurements in both model and prototype and the good agreement obtained in previous investigations, it is concluded that the effect of differences in Poisson's ratios between model and prototype is of little more than academic interest. It was concluded by Elstener (34) that ignoring the differences in Poisson's ratio does not seriously affect the comparison between the model and the ACI Direct Design or Empirical Method.

5.2.2 FIGURE 5.3 THROUGH FIGURE 5.18

In these figures comparison is made between the moments given by the Direct Design Method (ACI 318-71) and the experimentally measured moments across different sections in both directions for each level. For each negative (column line) section there is a corresponding positive (middle line) section, e.g., Figure 5.3 and Figure 5.7. The ACI moments variations across the section are presented by straight lines, while the measured moments are presented by curves. On these curves the location of the moment on the cross section is plotted against the value of that moment.

It can be seen from these figures that there are three different pairs of curves (direct design method moments versus the measured moments of the model of this investigation).

a) One pair of curves represent moment values and variation across a section due to a uniformly distributed load on all panels. This will be used in a comparison between the measured moments and the direct design moments of the

ACI 318-71, Section 13.3.2 through section 13.3.4.5.

b) The second pair of curves represent the maximum negative moment values and variation across a section, due to strip loading patterns with the live load equal to 3.64 times the dead load, versus 1.11 (1/0.9) the direct design negative moments. This pair of curves will be used in a discussion of the effects of pattern loadings, within the limits of the direct design method when the live load does not exceed three times the dead load (all loads are due to gravity only and uniformly distributed over an entire panel).

c) The third pair of curves represent the maximum positive (or the corresponding negative) moment values and variation across a section, due to strip loading patterns with the live load equal to 3.64 the dead load, versus the direct design positive (or the corresponding negative) moment values with Provisions 13.3.6. This pair of curves will be used in a discussion of Provisions 13.3.6 and of the effects of pattern loadings on the direct design method for the above mentioned conditions.

5.2.3 TABLE 5.1 THROUGH TABLE 5.8

Comparison is made, in these tables, between the direct design method moments (ACI 318-71), the empirical method moments (ACI 318-63) and the experimentally measured moments (positive and negative for both column and middle strips).

It was immediately noted that the total panel moments of the model from this investigation are approximately from seven to zero percent lower than the total static design moment "M_o" by ACI 318-71. Errors in the constants necessary to calculate moments from strains and experimental error could account for this small difference.

The value of ACI 318-71 static moment "M_o" was accepted as a reference value, and the ratio between every moment (by the direct design method, empirical method and experimentally measured moments) and M_o was calculated.

Every table is divided into two main parts:

a) ACI moments, and

b) Experimentally measured moments.

a) ACI Moments:

In this part of the tables there are three sets of numbers. The first set of numbers without brackets represents values of moments calculated by the direct design method (ACI 318-71), e.g. Line 1, Table 5.1. The second set of numbers with brackets, and asterisks, represents values of moments calculated by the empirical method (ACI 318-63), e.g. Line 2, Table 5.1. The third set of numbers with brackets, without asterisks, represents the empirical method moments as a percentage of the reference moment "M_o", e.g. Line 5, Table 5.1.

b) Experimentally Measured Moments

In this part of the tables the values of the moments were obtained from the curves of Figure 5.3 through Figure 5.18 for the case of all panels loaded. As previously mentioned, M_o refers to the reference moment.

5.3 DISCUSSION OF RESULTS

In this discussion two main points will be considered:

a) Moment values and variation in a flat slab due to uniformly distributed load on all the panels, and

b) The effect of pattern loadings on the moments, when the uniformly distributed live load does not exceed three times the uniformly distributed dead load.

Section 5.4 discusses the moments in an interior panel, and Section 5.5 discusses the moments in an end span interior panel.

For each panel the discussion will be conducted as follows:

1. The total panel moment (negative moment + positive moment)
2. The total panel negative moment.
3. The total panel positive moment.
4. The total column strip moment.
5. The total middle strip moment.

5.4 INTERIOR PANEL (P5)[1]

See Table 5.1 through Table 5.4 and Figure 5.3 through Figure 5.10.

It is immediately noted that the measured total interior panel moments (negative moment + positive moment) of the model of this investigation are approximately from 5 percent lower to 2 percent higher

[1]All discussions for interior panel is made in parallel between the Direct Design Method (ACI 318-71) and the Empirical Method (ACI 318-63).

than the total static moment M_o given by the direct design method (4). Errors in the constants necessary to calculate the moments from the strains, and experimental errors could account for this small difference.

5.4.1 The Total Panel Moments {M(-ve) + M(+ve)} Calculated by the Direct Design Method are Approximately:

a) From 6.5 to 8 percent higher than the total static moment (M_o)[2],

b) 12, 5, 11 and 16 percent higher than the total measured moments[3] (Table 5.1 to Table 5.4 respectively)[4],

c) 25, 26, 25 and 26 percent higher than the total empirical method moments for an interior panel.

The total calculated static moment "M_o"[5] is, approximately: 5 percent higher, 2 percent lower, 3 percent higher and 7 percent higher than the total measured moments.

If Section 13.3.4.6[6] (ACI 318-71), can be applied it would reduce the total panel moments, calculated by the direct design method to the total static moment "M_o". This reduction can be achieved by reducing the negative moment, only, to 65% M_o.

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[2] See Fig. 13.4 of Chapter 13 of Commentary on Building Code Requirements for Reinforced Concrete (ACI 318-71).

[3] The measured moments of the model of this investigation will be called "the measured moments" throughout this discussion.

[4] All the comparisons for interior panels will be made in that sequence (i.e. Table 5.1 to Table 5.4, respectively).

[5] Moments calculated by the direct design method, of ACI 318-71, will be called "the calculated moments" throughout this discussion.

[6] 13.3.4.6. A design moment may be modified by 10 percent provided the total static design moment for the panel in the direction considered is not les than that required by Eq. 13.2.

In the above discussion it can be seen that there is good agreement between the direct design method and the model test results for the value of the total panel moment.

5.4.2 The Total Interior Negative Moment of an End Span (i.e. the exterior moment of the first interior span), Calculated by the Direct Design Method is Approximately:

a) 19, 15, 5 and 16 percent higher than the total measured moments. This represents a difference of 11.4, 9.3, 3.4 and 9.9 percent of "M_o". This difference is split between the column and middle strips as follows:[7]

	Total	Column Strip	Middle Strip
First level (Y-direction)	+11.4% M_o	+ 8.9% M_o	+2.5% M_o
First level (X-direction)	+ 9.3% M_o	+11.4% M_o [8]	-2.1% M_o [8]
Second level (Y-direction)	+ 3.4% M_o	+ 5.6% M_o	-2.2% M_o [9]
Second level (X-direction)	+ 9.9% M_o	+15.8% M_o [8]	-5.9% M_o [8]

[7]Positive sign will be used for a positive difference and negative sign for a negative difference (calculated moments-measured moments).

[8]ACI 318-71, Sections 13.1.2 and 13.1.3 state that:
13.1.2 A column strip is a design strip with width of 0.25 ℓ_2 but not greater than 0.25ℓ_1 on each side of the column centre line. The strip includes beams, if any.
13.1.3 A middle strip is a design strip bounded by two column strips.
The above, two sections, result in some concentration of design moments in the short direction, of rectangular panels, in the column strip.

[9]This negative difference could be due to experimental error.

b) 30, 32, 32 and 33 percent higher than the empirical method moments.

It can be seen that the direct design method agrees with, or underestimates, the test model results for the negative moment for a middle strip, while over-estimates the value of the negative moment for a column strip.

Now, if the previously suggested reduction (of almost 10%) in the negative moment was distributed, between the column and the middle strips, it would decrease the gap, between the calculated moments and the measured moments, for the column strip, and increase it for the middle strip.

If the whole difference is applied to the column strip values only, the following results would be obtained.

a) The total, direct design, negative moments will become: 721, 621, 721 and 621 which are; 8 percent higher, 3 percent higher, 5 percent lower and 3 percent higher than the total measured moments.

b) The middle strip, direct design, negative moments will remain: 15 percent higher, 10 percent lower, 11 percent lower[10], and 24 percent lower than the measured moments.

c) The column strip, direct design, negative moments will become: 423, 449, 520.5 and 447, which are; 5, 10, -3 and 20 percent higher than the measured moments.

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[10] As mentioned before, this could be due to an experimental error.

5.4.3 The Total Calculated Positive Moment of an Interior Span is Approximately:

a) Equal, 12 percent lower, 26 percent higher and 17 percent higher than the total measured moments.

This difference is represented in the following table as a percentage of M_o:

	Total	Column Strip	Middle Strip
First level (Y-direction)	+0.1% M_o	+2.9% M_o	-2.8% M_o
First level (X-direction)	-4.9% M_o	+2.5% M_o	-7.4% M_o
Second level (Y-direction)	+7.2% M_o	+6.4% M_o	+0.8% M_o
Second level (X-direction)	+5.2% M_o	+6.9% M_o	-1.6% M_o

From the above table it can be seen that, the calculated positive moments overestimate the measured positive moments for the column strip and under estimate the measured positive moments for the middle strip.

From Figure 5.7 through Figure 5.10 (which show a comparison of the calculated positive moments and the measured positive moments across a section for various loading patterns) it can be seen that, the measured positive moment for a panel is almost uniformly distributed across the width of the panel.

b) From 14 to 15 percent higher than the empirical method moments.

5.4.4 The Total Column Strip Moment of an Interior Span, Calculated by the Direct Design Method (line 6, Tables 5.1 to 5.4) is Approximately:

a) 18.8, 22.5[8], 18.9, and 42.5[8] percent higher than the total measured moments (line 15, Tables 5.1 to 5.4).

b) 28.5, 48.5[11], 29.5 and 49.6[11] percent higher than the total empirical method column strip moments. Now, if the previously mentioned reduction to the calculated column strip negative moment is applied the following results would be obtained:

- The total calculated column strip moment of an interior span is: 8.4, 10.8[8], 7.3 and 27.5[8] percent higher than the total measured moment for a column strip.

5.4.5 The Total Middle Strip Moment of an Interior Span, Calculated by the Direct Design Method (e.g. line 6, Tables 5.1 to 5.4) is Approximately:

a) 1, 23[8] and 19[6] percent lower than the total measured moments of the model of this investigation (line 15, Tables 5.1 to 5.4).

b) 16 percent higher, 8 percent lower, 17 percent higher and 7 percent lower than the total empirical method middle strip moment.

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[11] In the empirical method (ACI 318-63) the column strip in either direction was considered to be that portion of the slab bounded by the quarter panel on each side of the column line and the middle strip consisted of the remaining central half panel bounded by two column strips.

5.5 END SPAN (P2 and P8 in the Y-direction, and P4 and P6 in the X-direction)

Table 5.5 through Table 5.8,

Figure 5.11 through Figure 5.18, and

Figure 5.19 through Figure 5.22.

It is immediately noted (from Table 5.5 through Table 5.8) that the empirical method is far from accurate (especially for exterior negative moments) for an end span. Therefore, it will not be considered in this discussion.

5.5.1 The Total Calculated Panel Moment (M_o) is Approximately:

a) 9.3, 2.3, 5.8 and 4.1 percent higher than the total measured moments (Figure 5.19 to Figure 5.22, respectively)[12].

It can be noted that there is a very good agreement between the direct design method and the model test results for the value of the total panel moment.

Errors in the constants necessary to calculate the moments from the measured strains, and experimental errors could account for this small difference. On average the calculated panel moment is 5.4 percent (7.5 in y-direction and 3.2 in x-direction) higher than the measured moment.

[12]All the comparisons, in the following discussion, for end spans will be done in the sequence, Figure 5.18 to Figure 5.22.

5.5.2 THE TOTAL EXTERIOR NEGATIVE MOMENT OF AN END SPAN

The Code overestimates the value of the exterior negative moment for the column strip.

The value of the exterior negative moment for the middle strip is almost zero, however the measured moments show a small positive value due to the edge strip (3.25 in. wide all around).

Further tests were carried out with strain gages mounted on the edge of the slab (in the middle strip) and the measured moments at the edge in the direction perpendicular to the edge were found to be zero.

5.5.3 THE TOTAL CALCULATED POSITIVE MOMENT OF AN EXTERIOR SPAN IS APPROXIMATELY:

10, 14, 9 and 13 percent lower than the measured moments. The difference between the two moments (calculated moments - measured moments) is represented in the following table as a percentage of M_o. In the same table the value of "the calculated moments ÷ the measured moments" is represented between brackets.

	Total	Column Strip	Middle Strip
First level (y-direction)	-5.7% M_o (0.90)	+2.6% M_o (1.09)	-8.3% M_o (0.72)
First level (x-direction)	-9.4% M_o (0.86)	+3.9% M_o [8] (1.13)	-13.3% M_o [8] (0.63)
Second level (y-direction)	-5.1% M_o (0.91)	+3.1% M_o (1.10)	-8.2% M_o (0.73)
Second level (x-direction)	-8.8% M_o (0.87)	+4.8% M_o [8] (1.16)	-13.6% M_o [8] (0.63)

From the above table it can be seen that, the calculated positive moments overestimate the measured positive moments for the column strip, and under estimate the measured positive moments for the middle strip.

It must also be noted that the total calculated positive moment under estimates the total measured positive moment.

From Figure 5.15 through Figure 5.18 (which show a comparison of the calculated positive moments and the measured positive moments across a section for various loading patterns) it can be seen that, the measured positive moment for a panel is almost uniformly distributed across the width of the panel.

Now, if the previously suggested reduction (of almost 10%) in the interior calculated negative moment was applied (see the discussion of the interior panel P5) and the calculated positive moment was increased by the same amount (to keep the total calculated panel moment equal to M_o, which is in a good agreement with the total measured panel moment), and if this moment is assumed to be distributed uniformly across the cross section (the middle strip + the column strip) the following results should be obtained.

a) The total calculated positive moments will become: 662, 596, 697 and 622 which are: 1 percent higher, 3 percent lower, 3 percent higher and 1 percent higher than the total measured positive moments.

b) The column strip calculated positive moments would become: 331, 261, 348 and 272 which are: 2 percent higher, 6.8 percent lower, 3.5 percent higher and 3.3 percent lower than the column strip measured positive moments.

c) The middle strip calculated positive moments will become: 331, 335, 348 and 350 which are: 0.6 percent higher, 0.7 percent lower, 3.2 percent higher and 0.6 percent higher than the middle strip measured positive moments.

5.5.4 The total column strip moment of an end span, calculated by the direct design method (see Figure 5.19 to Figure 5.22) is approximately: 24.4, 25.6, 22.2 and 34.8 percent higher than the total measured moments.

If the previously mentioned reduction is now applied to the calculated column strip interior negative moment, the following results would be obtained.

- The total calculated column strip moment of an end span would be 18.7, 19.0, 15.7 and 27.0 percent higher than the total measured moment.

5.5.5 The total middle strip moments of an end span, calculated by the direct design method (see Figure 5.19 to Figure 5.22) are approximately: 14.2, 26.9[8], 17.9 and 29.4[8] percent lower than the total measured moments.

5.6 EFFECT OF PATTERN LOADINGS

This effect is considered within the Direct Design Method limitations (i.e. the uniformly distributed live load does not exceed three times the uniformly distributed dead load).

5.6.1 MAXIMUM NEGATIVE MOMENT PATTERNS

From Figure 5.3 to Figure 5.6 and Figure 5.11 to Figure 5.14 it can be seen that the Direct Design Method values for the maximum negative moments are comparable to the measured maximum negative moments due to the appropriate loading pattern. Thus no provisions are required for the effect of pattern loadings on the negative moments.

5.6.2 MAXIMUM POSITIVE MOMENT PATTERNS

See Figure 5.7 to Figure 5.10 and Figure 5.15 to Figure 5.18. It can be seen that the Direct Design Method provision "13.3.6" is not sufficient for the value of the positive moment in the middle strip.

5.6.3 REVERSAL OF STRESSES

Dead load on all panels plus live load on panels P_1, P_4, P_7, P_3, P_6 and P_9 would tend to give a negative moment at the mid span of the interior panel P_5 (strain gage no. 50) in the x-direction. Similarly, live loads on panels P_1, P_2, P_3, P_7, P_8 and P_9 tends to give negative moments at the mid span of panel P_5 (strain gage no. 50) in the y-direction. The magnitude of these interior panel negative moments depend upon the relative magnitudes of the live to dead load. Considering the extreme case where the live load is three times the dead load (load factors of 1.7 for live load and 1.4 for dead load need to be considered), the extreme negative moments are a large proportion of the corresponding design positive moments, namely, -11.37 cf 19.7 and -10.65 cf 28.6 for M_x and M_y, respectively, for level 1. Similarly, for level 2 the comparison shows values of M_x of -15.04 cf 20.1 and for M_y of -16.59 cf 29.7.

Alternatively, the ratio of factored live load to factored dead load not to cause negative moments can be calculated as below

First Level

D.L. = 86.2 psf

M_x = 4.16 lb.in.

M_y = 5.65 lb.in.

L.L. = 313.8 psf on panels P1, P3, P4, P6, P7, P9

M_x = -15.53 lb.in

L.L. = 313.8 psf on panels P1, P2, P3, P7, P8, P9

M_y = -16.30 lb.in.

Extreme values:

Negative values:

$M_x = -11.37\ (0.00)^{13}$

$M_y = -10.65\ (0.00)^{13}$

Positive values:

$M_x = 4.16 + 30.69 = 34.85\ (19.7)^{13}$

$M_y = 5.65 + 36.87 = 42.52\ (28.6)^{13}$

<u>Second Level</u>

D.L. = 86.2 psf

$M_x = 3.21$ lb.in.

$M_y = 4.25$ lb.in.

L.L. = 313.8 psf on panels P1, P3, P4, P6, P7, P9

$M_x = -18.25$ lb.in.

L.L. = 313.8 psf on panels P1, P2, P3, P7, P8, P9

$M_y = -20.84$ lb.in.

Extreme values:

Negative values:

$M_x = -15.04\ (0.00)^{13}$ lb.in.

$M_y = -16.59\ (0.00)^{13}$ lb.in.

Positive values:

$M_x = 3.21 + 29.94 = 33.15\ (20.1)^{13}$ lb.in.

$M_y = 4.25 + 36.32 = 40.57\ (29.7)^{13}$ lb.in.

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[13] Moments calculated by ACI 318-71 "Direct Design Method", with provisions for effects of pattern loadings taken into consideration.

Assume: D.L. = D psf

L.L. = L psf

design D.L. = 1.4 D

design L.L. = 1.7 L

Thus we obtain the following experimental moments values for the First and Second Levels.

First Level

Due to a design D.L. = 1.4 D moments are

M_x = .0676 D

M_y = .0917 D

Due to a design L.L. = 1.7 L maximum negative moments are

M_x = -0.08415 L

M_y = -0.0882 L

The extreme negative moments are

M_x = +0.0676 D - 0.08415 L

M_y = +0.0917 D - 0.0882 L

Second Level

Due to a design D.L. = 1.4 D moments are

M_x = 0.0521 D

M_y = 0.0691 D

Due to a design L.L. = 1.76 maximum negative moments are

M_x = -0.0989 L

M_y = -0.1129 L

The extreme negative moments are

$M_x = +0.0521\ D - 0.0989\ L$

$M_y = +0.0691\ D - 0.1129\ L$

Hence for a ratio of live load to dead load of .80 for level 1 and 0.60 for level 2 will not cause negative in nominally positive moment regions. The different values for the two levels is due to the differing relative column stiffnesses for the two levels.

CHAPTER 6

CONCLUSIONS

The following conclusions on the ACI 318-71 Direct Design Method are based on the results obtained from the tests conducted on the model, which contained four independent sets of results (X-Direction First Level, Y-Direction First Level, X-Direction Second Level and Y-Direction Second Level).

The conclusions are summarized as follows:

a) The total static moment given by the ACI 318-71 Direct Design Method is accurate for all practical purposes.

b) The width of the column strip specified by the ACI Code in Clause 13.1.2 of 0.25 ℓ_2 but not greater than 0.25 ℓ_1 results in too great a concentration of negative moment in the column strip. This concentration of moment in the column strip is even less justified for the positive moment where the column strip moment/unit width and the middle strip moments/unit width are of approximately equal magnitude.

c) The effect of the concentration of positive moment in the column strip together with aforementioned restriction of the width of the column strip results in middle strip design positive moments 25% less than those measured.

d) The provision "13.3.4.6" that a design moment may be adjusted by up to 10% provided that the total static moment is not less than required should be changed by eliminating the 10%

limit. It is recommended that adjustment be applied to the negative moment only.

e) The provisions for the effects of pattern loading "13.3.6" are insufficient.

f) The limitation of live load to dead load ratios, of "3:1", "13.3.1.5" is too high and results in a possibility of developing negative moments in or near mid span of interior panels.

The above stated conclusions cannot be translated into recommendations because not enough parameters were taken into consideration. Test series are needed to study the following:

- The bending moment values for an interior panel of a 3 bay x 5 bay model,
- The effect of the equivalent column flexural stiffness to the combined flexural stiffness of the slabs,
- The effect of the ratio of the longer to shorter spans within a panel for the values 1.5 and 2.0,
- The effect of differences in the successive span lengths (the successive span lengths need not differ by more than one-third of the longer span), and
- The effects of irregular layouts and boundary conditions.

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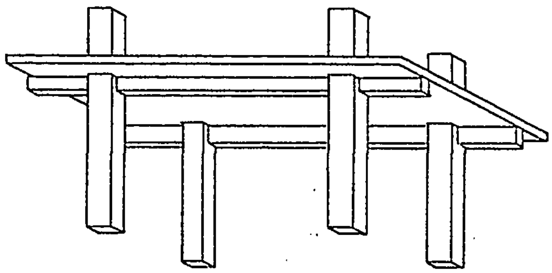
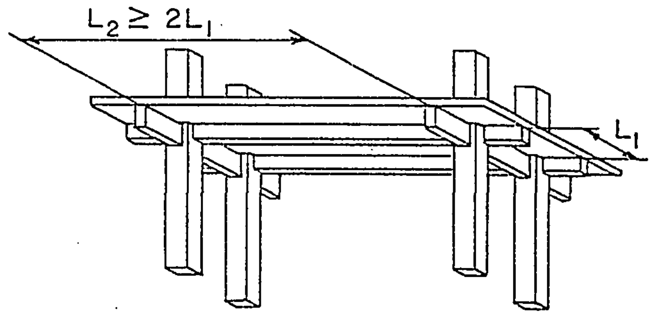
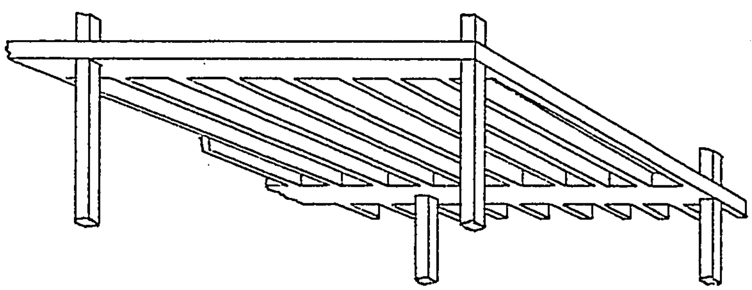
TABLES AND ILLUSTRATIONS



(A) ONE-WAY SOLID SLAB

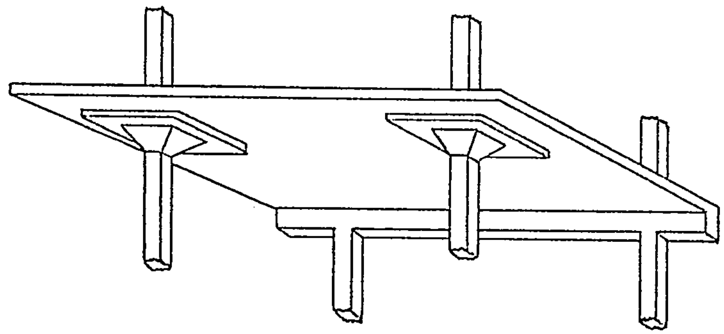


(B) ONE-WAY SOLID SLAB

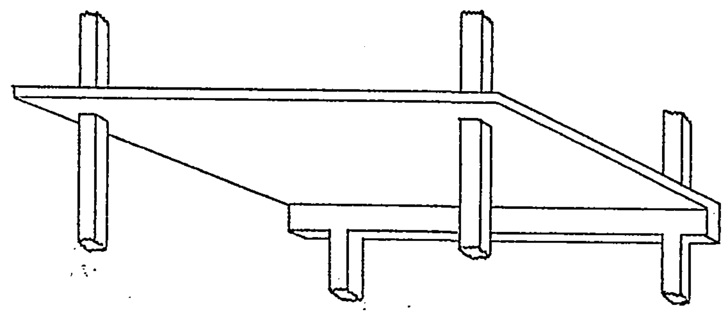


(C) ONE-WAY JOIST SLAB

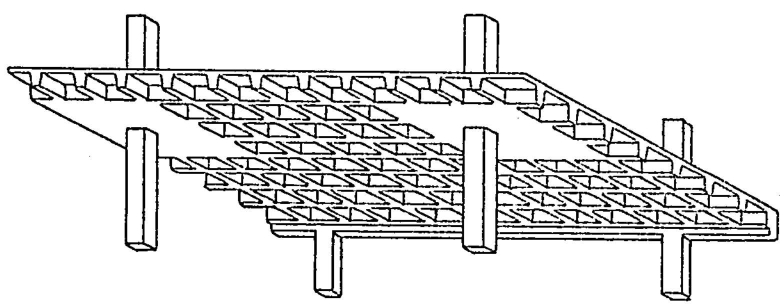
FIG. 2.1 : TYPES OF ONE-WAY SLAB SYSTEMS.



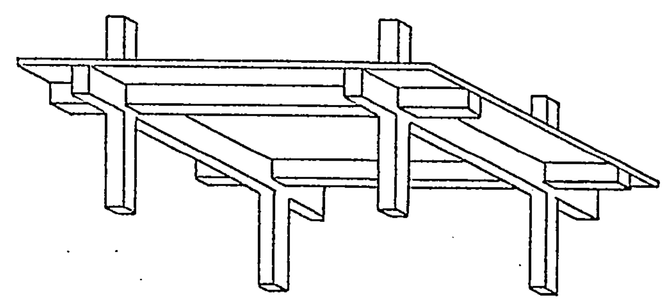
(A) SLAB WITHOUT BEAMS *
(FORMERLY FLAT SLAB)



(B) SLAB WITHOUT BEAMS *
(FORMERLY FLAT PLATE)



(C) WAFFLE SLAB
(FORMERLY WAFFLE FLAT PLATE)



(D) SLAB WITH BEAMS
(FORMERLY TWO-WAY SLAB)

FIG. 2.2: TYPES OF TWO-WAY SLAB SYSTEMS.

* TWO-WAY SLABS WITHOUT BEAMS MAY, OR MAY NOT, HAVE AN EDGE BEAM.

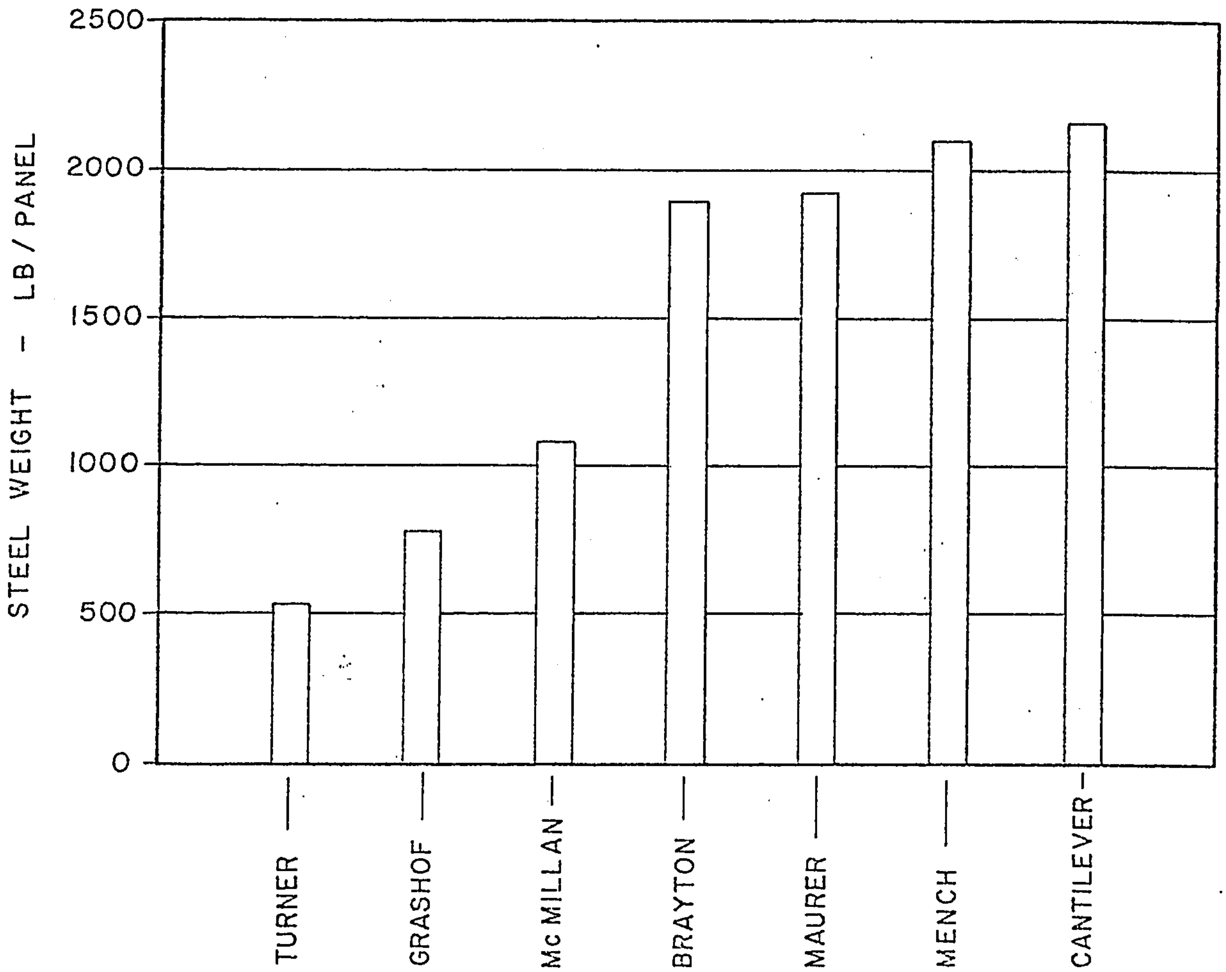


FIG. 2.3 WEIGHT OF STEEL REQUIRED IN THE INTERIOR PANEL OF A FLAT SLAB BY VARIOUS DESIGN METHODS IN 1910.

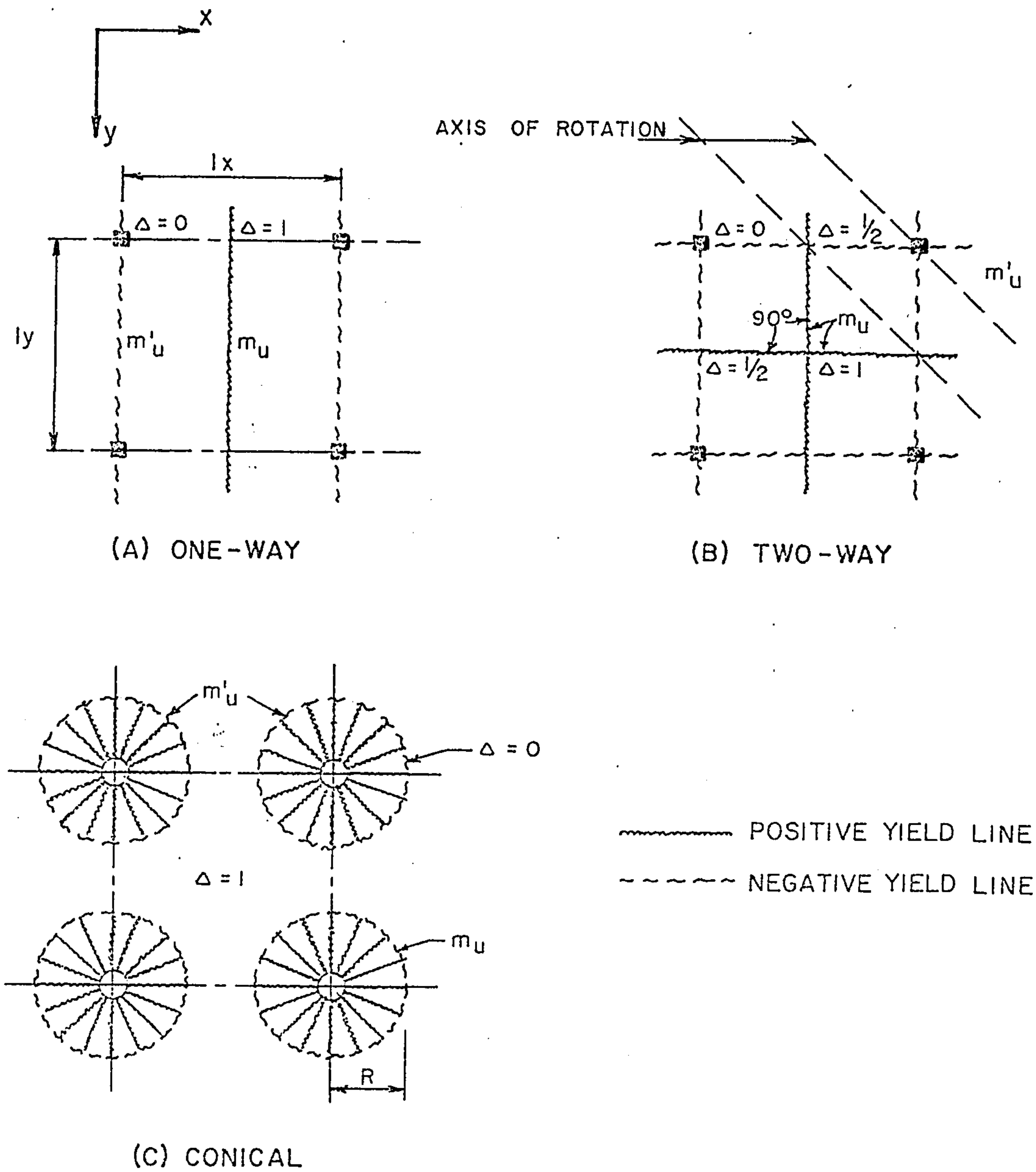


FIG. 3.1 COLLAPSE MODES OF FLAT-PLATES.

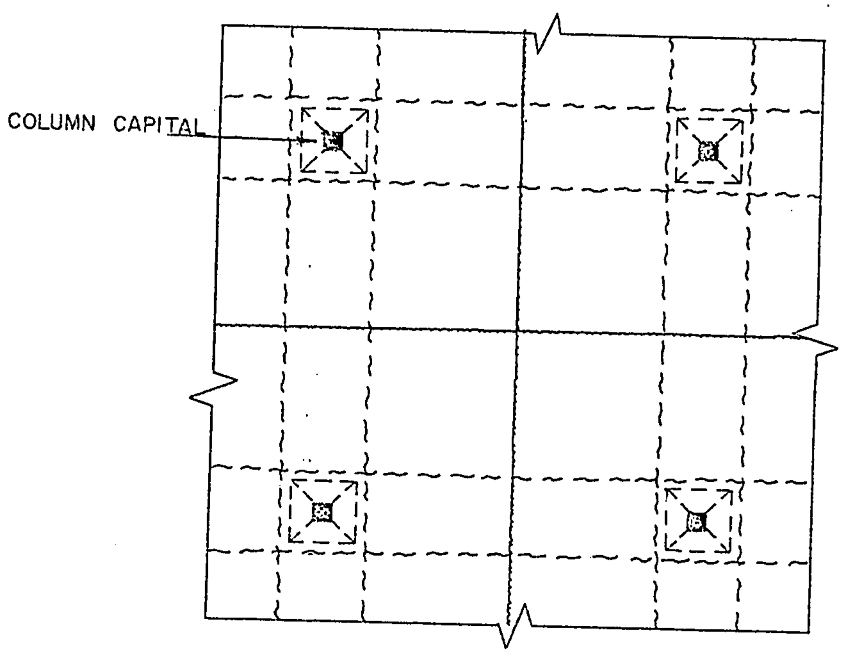


FIG. 3.2 YIELD - LINE PATTERN OF TYPICAL INTERIOR PANELS OF FLAT SLABS.

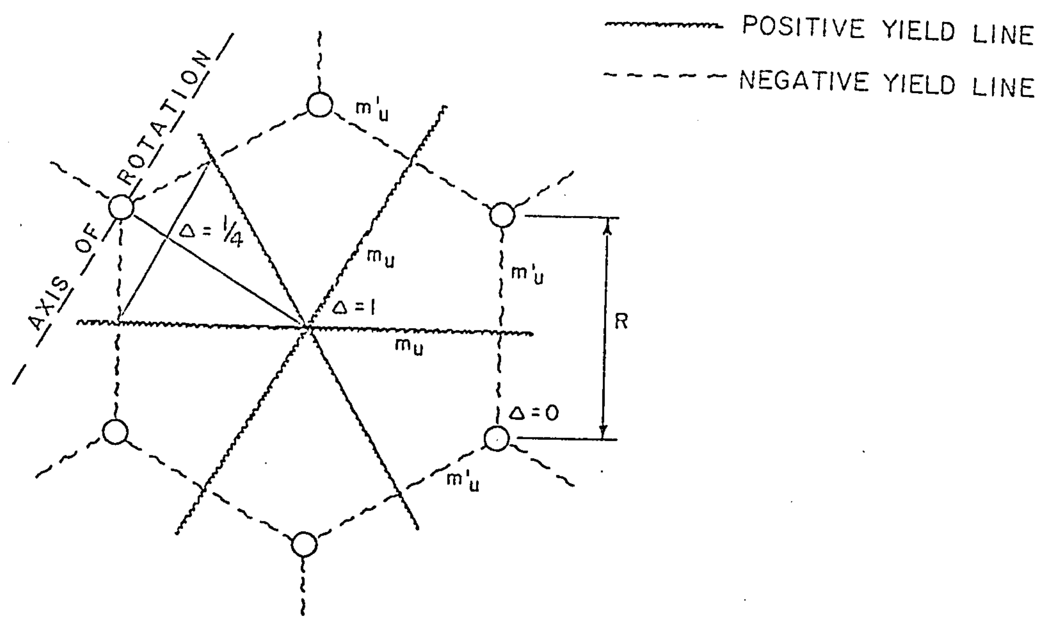
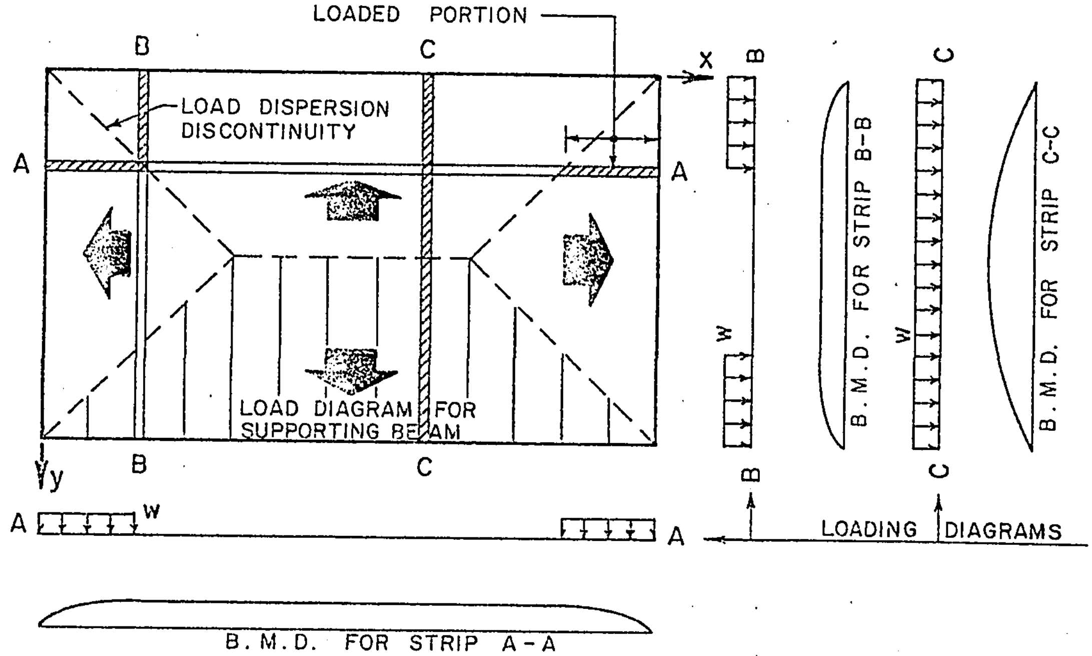
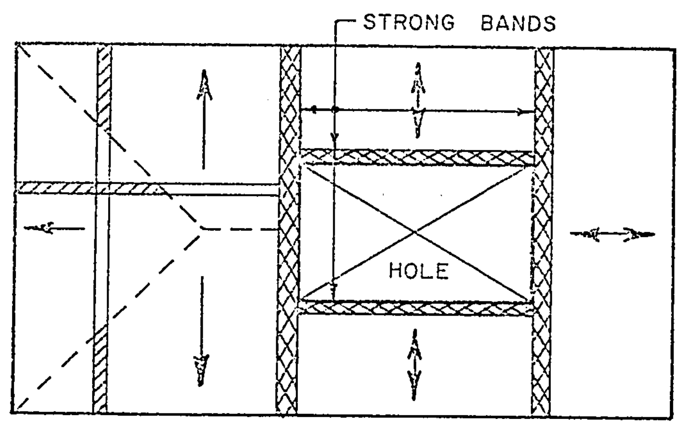


FIG. 3.3 COLLAPSE MODE OF A FLAT SLAB WITH HEXAGONALLY ARRANGED SUPPORTS.



(A) SIMPLE RECTANGULAR SLAB



(B) SLAB WITH A HOLE

FIG. 3.4 LOAD DISPERSION DIAGRAMS FOR STRIP METHOD

Property	Units	Mean Value	Standard Deviation	Remarks
Tensile strength	lb./sq.in. kg/cm^2	12,000 840	500 35	Straining rate: 1% per sec.
Flexural strength	lb./sq.in. kg/cm^2	20,000 1,400	1,000 70	
Shear strength	lb./sq.in. kg/cm^2	11,000 800	500 35	
Impact strength	ft.lb./in. of notch	0.32	-	
Impact strength	ft.lb.	0.2	-	
Young's Modulus* in flexure	lb./sq.in. kg/cm^2	4.67×10^5 3.34×10^4	0.15×10^5 0.1×10^4	Moduli in flexure, tension and comp. substantially the same
Pyramid hardness	-	22	-	
Poisson's ratio*	-	0.35	-	

TABLE 4.1 MECHANICAL PROPERTIES OF PERSPEX AT 68°F (20°C)

*Those values were measured at 70°F (21°C)

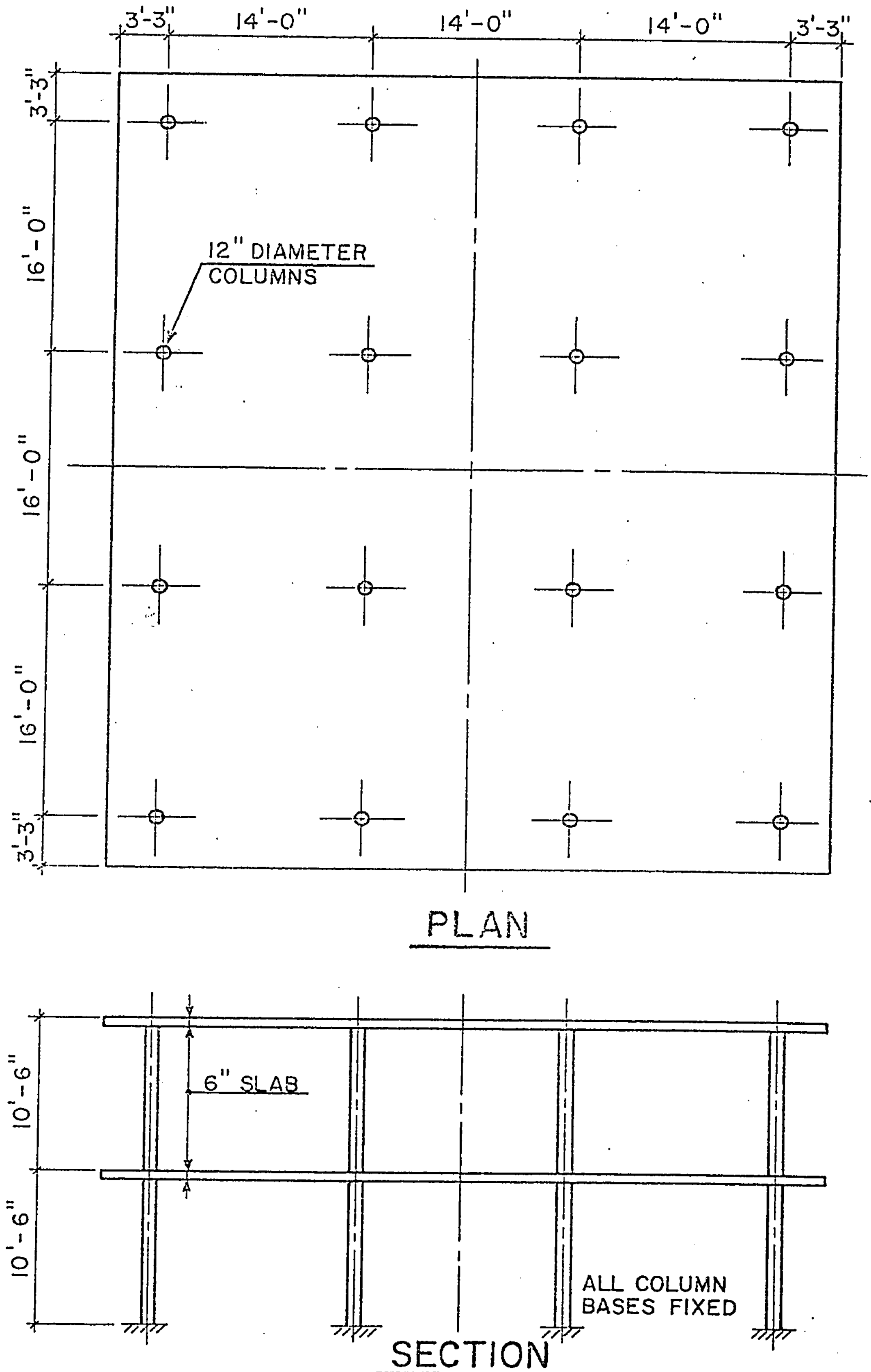
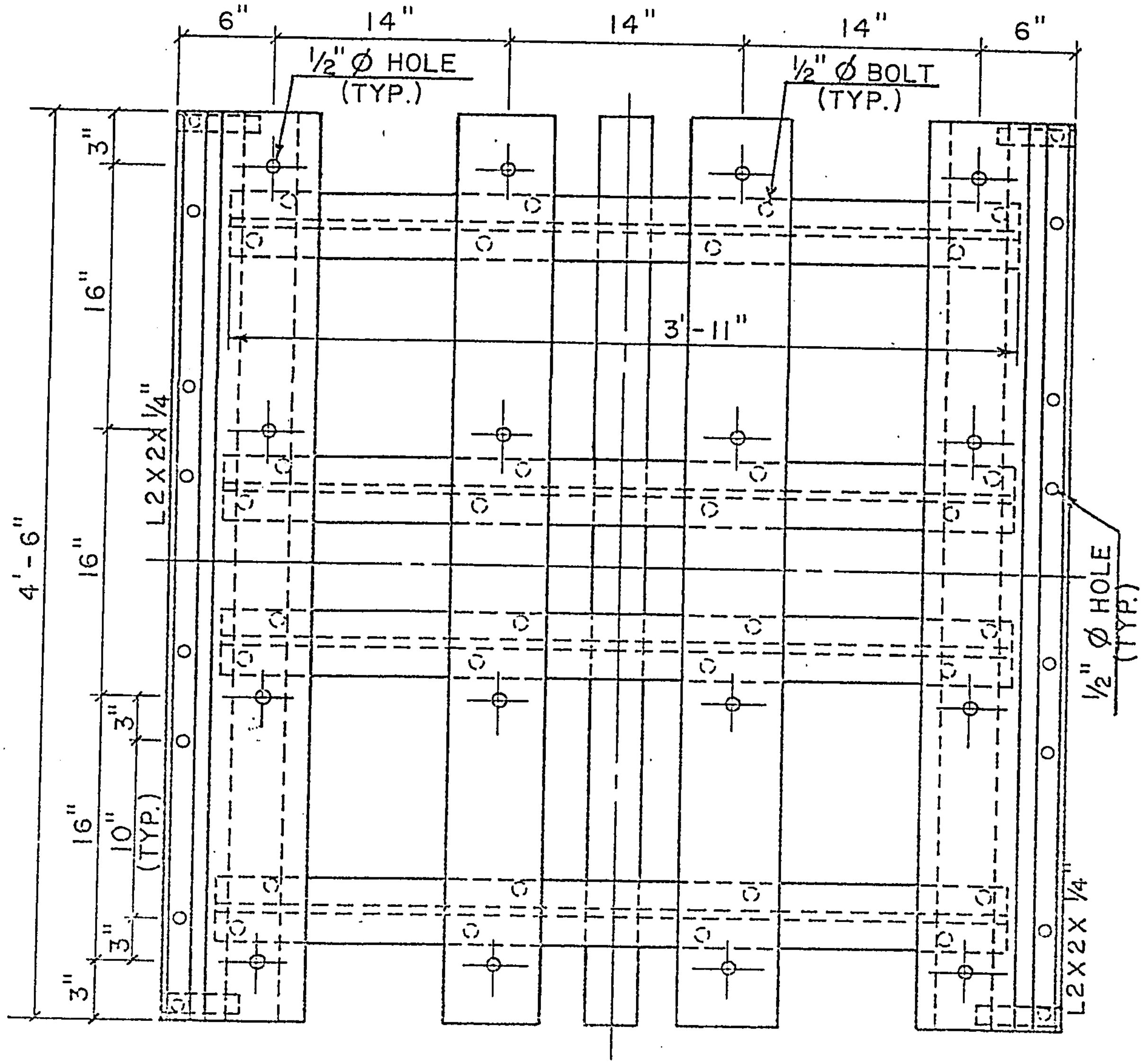
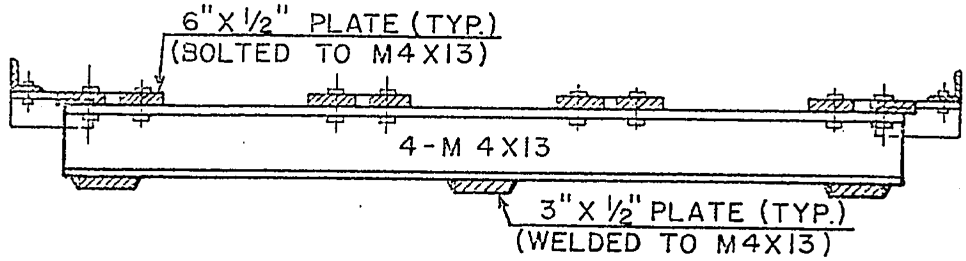


FIG. 4.1: LAYOUT OF THE PROTOTYPE



PLAN



SECTION

FIG. 4.2 : TEST FRAME LAYOUT

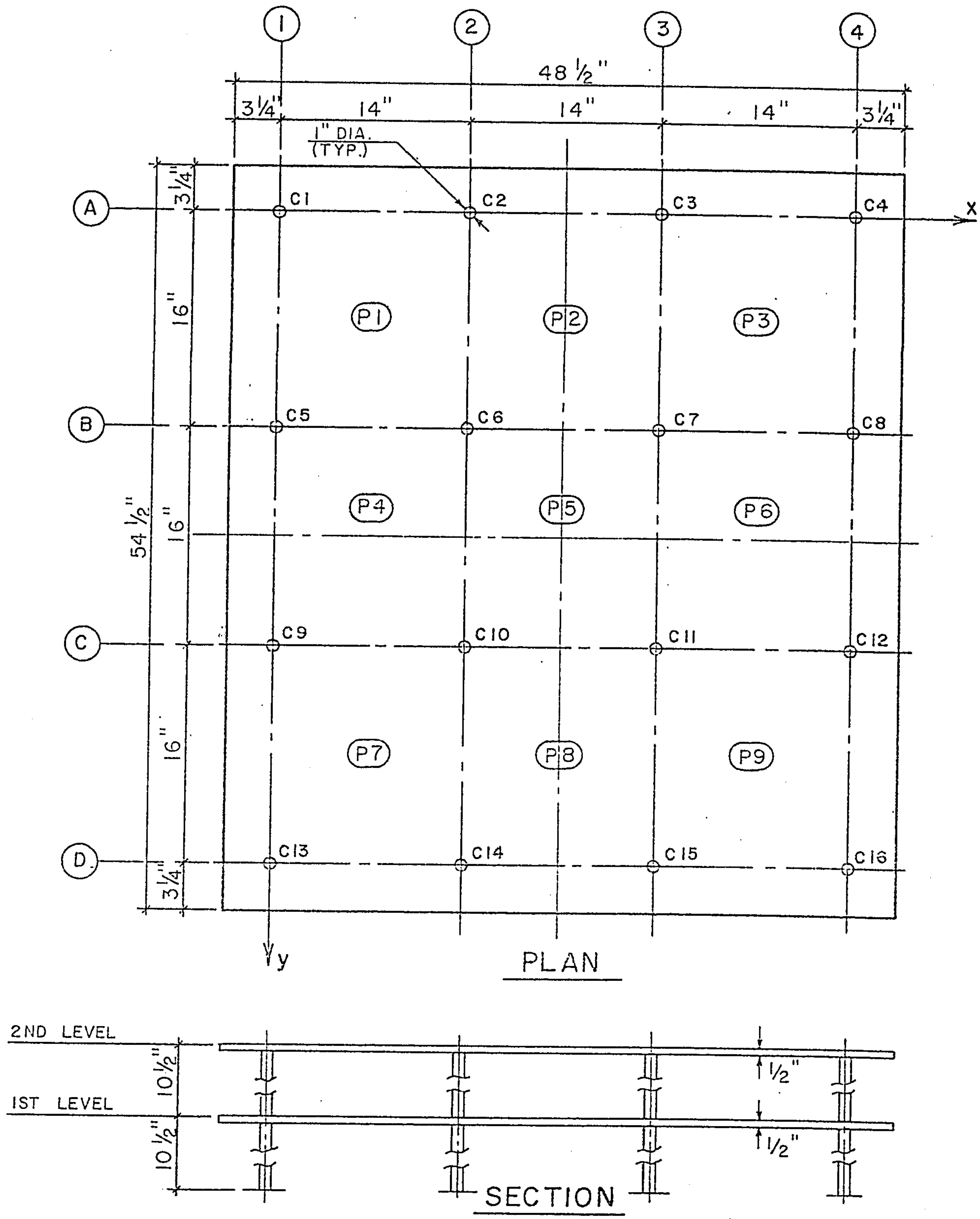


FIG. 4.3: LAYOUT OF MODEL

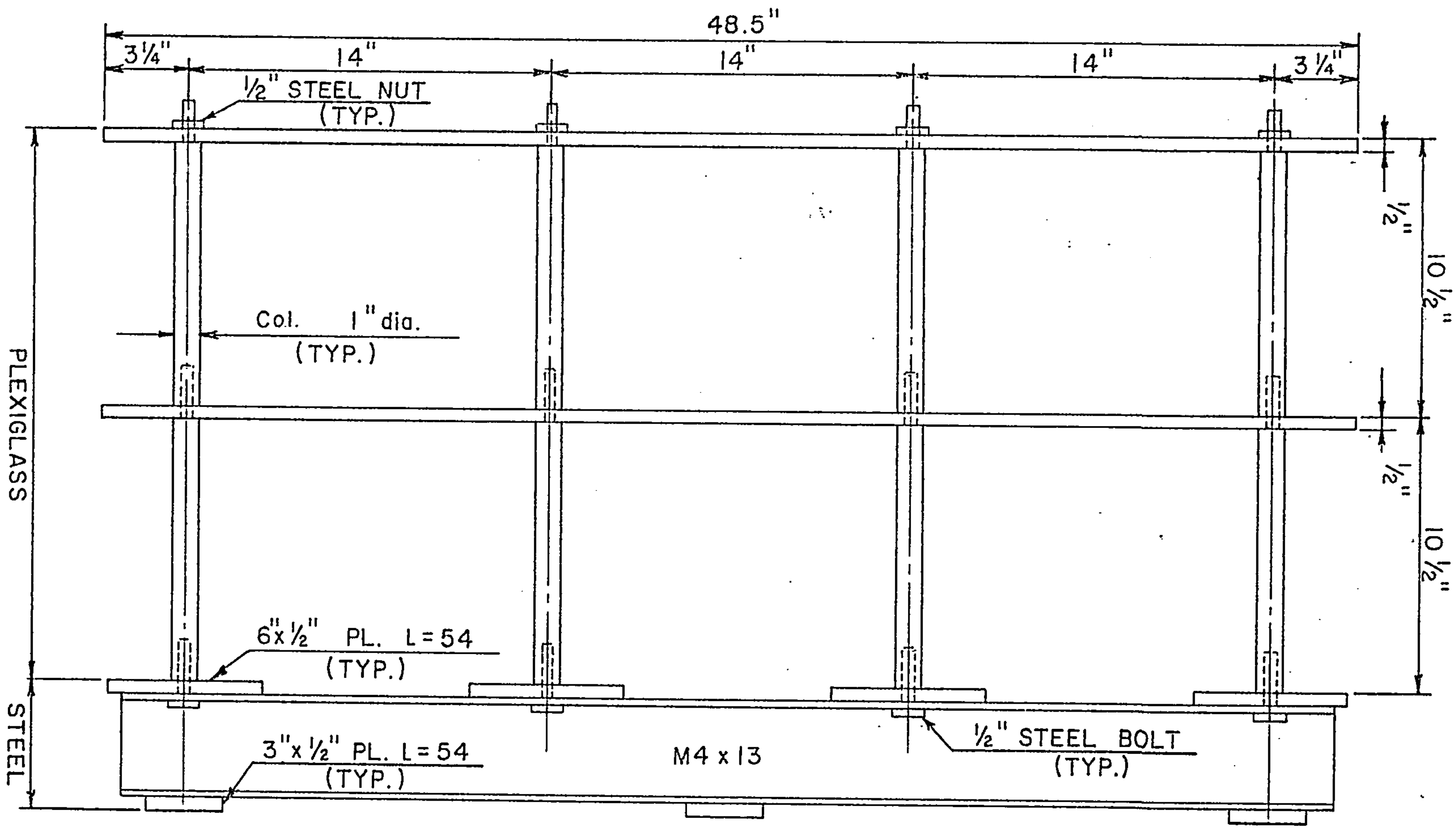


FIG. 4.4: ELEVATION OF TEST STUCTURE (IN THE X-DIRECTION)

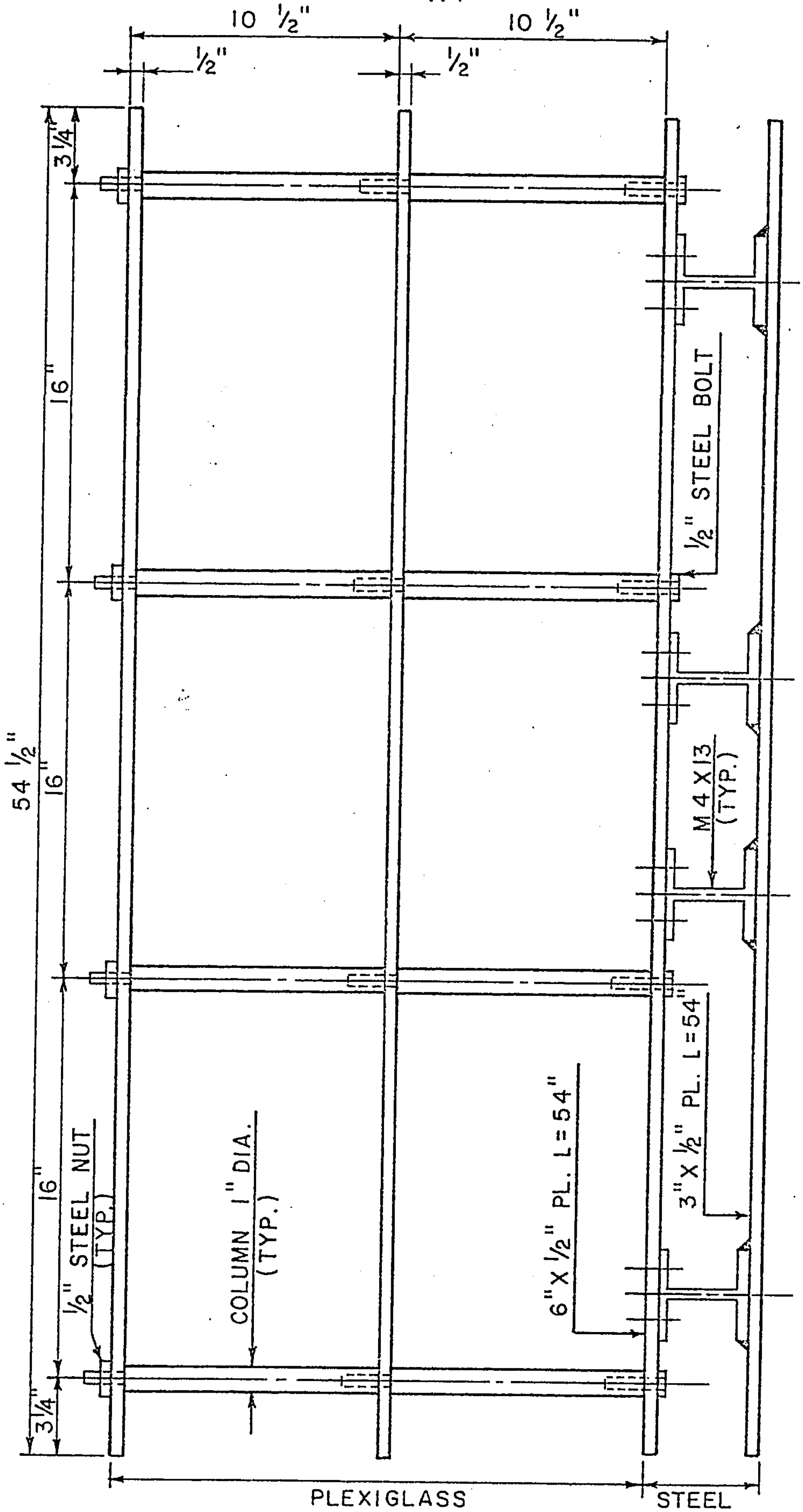
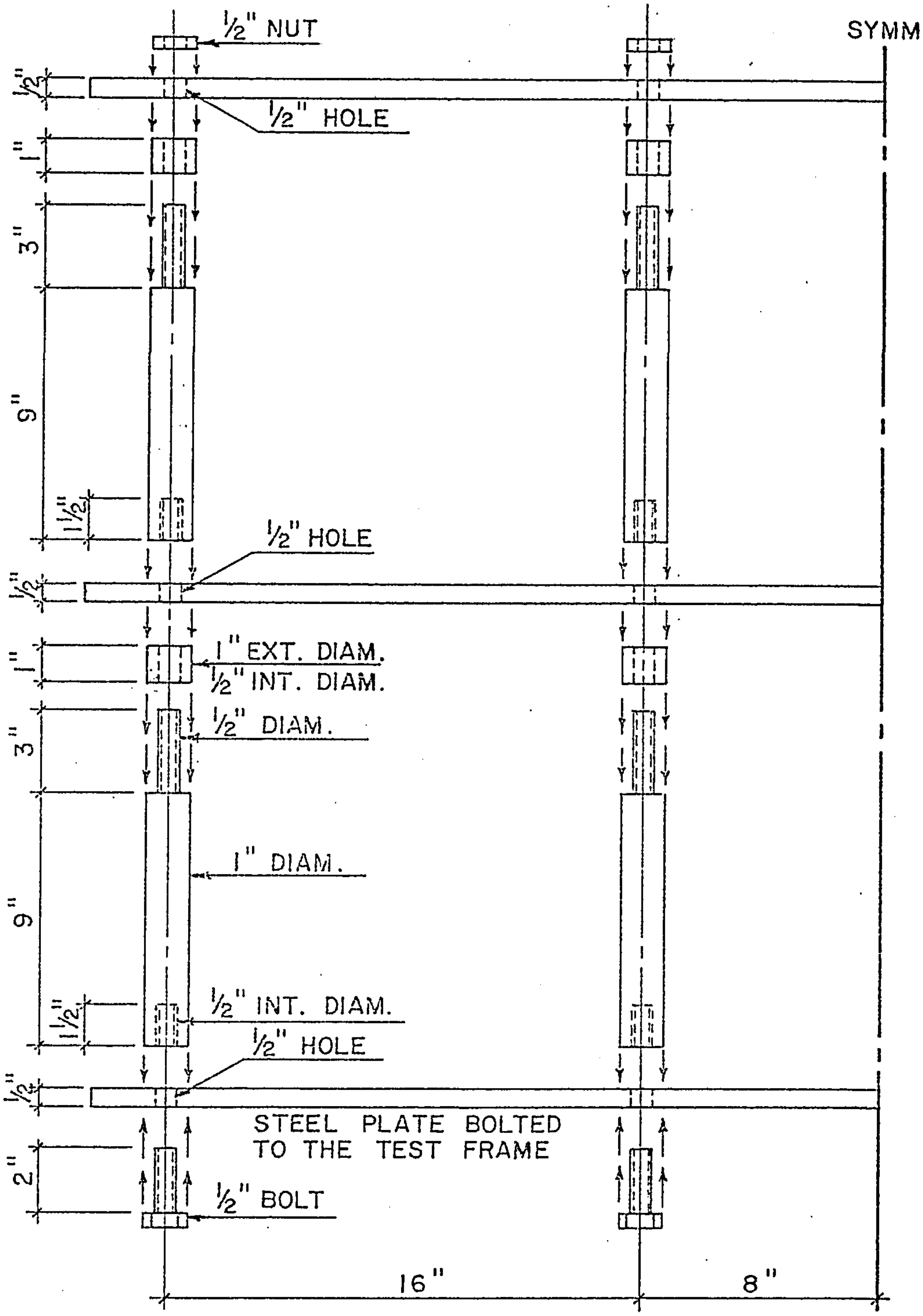


FIG. 4.5: ELEVATION OF TEST STUCTURE (IN THE Y-DIRECTION)



HALF ELEVATION

FIG. 4.6: MODEL COMPONENTS.

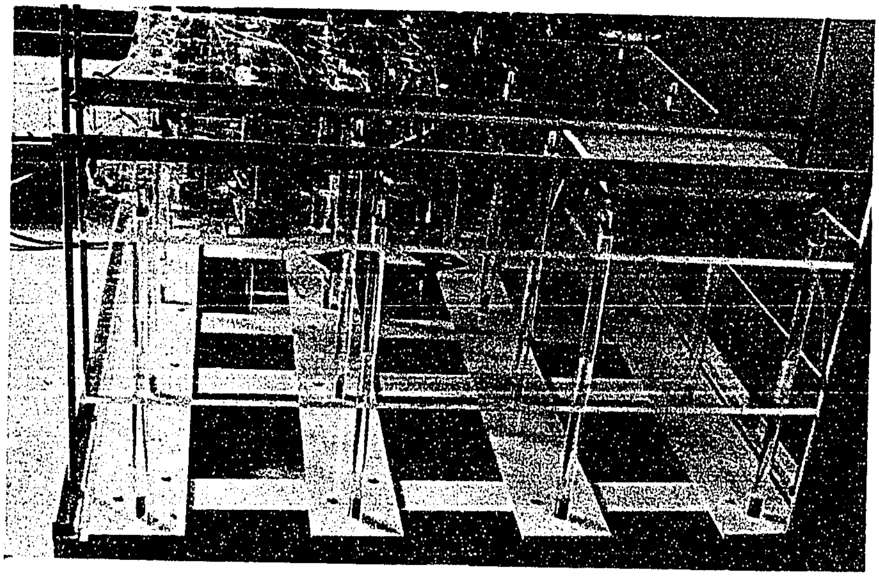


FIG. 4.7 THE TEST FRAME

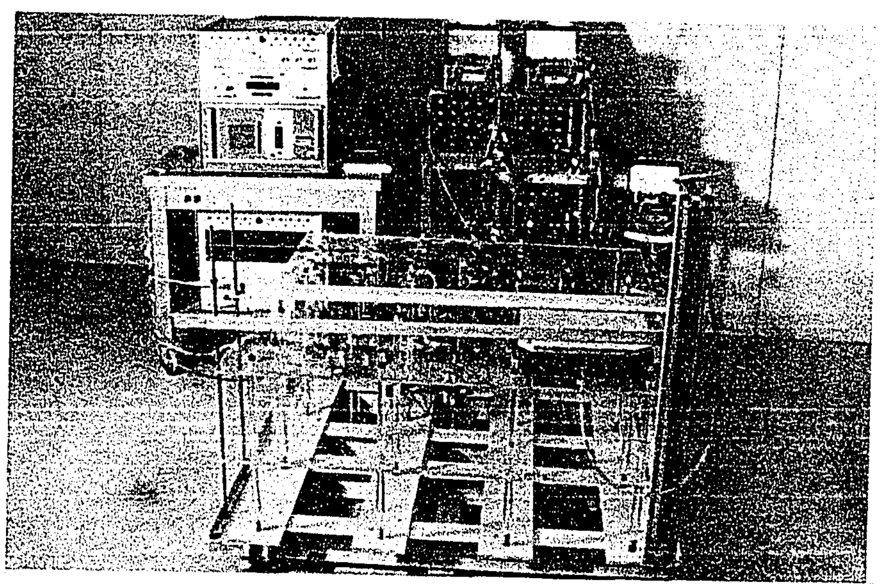


FIG. 4.8 THE MODEL AND THE INSTRUMENTS USED DURING THE TESTING OF THE MODEL

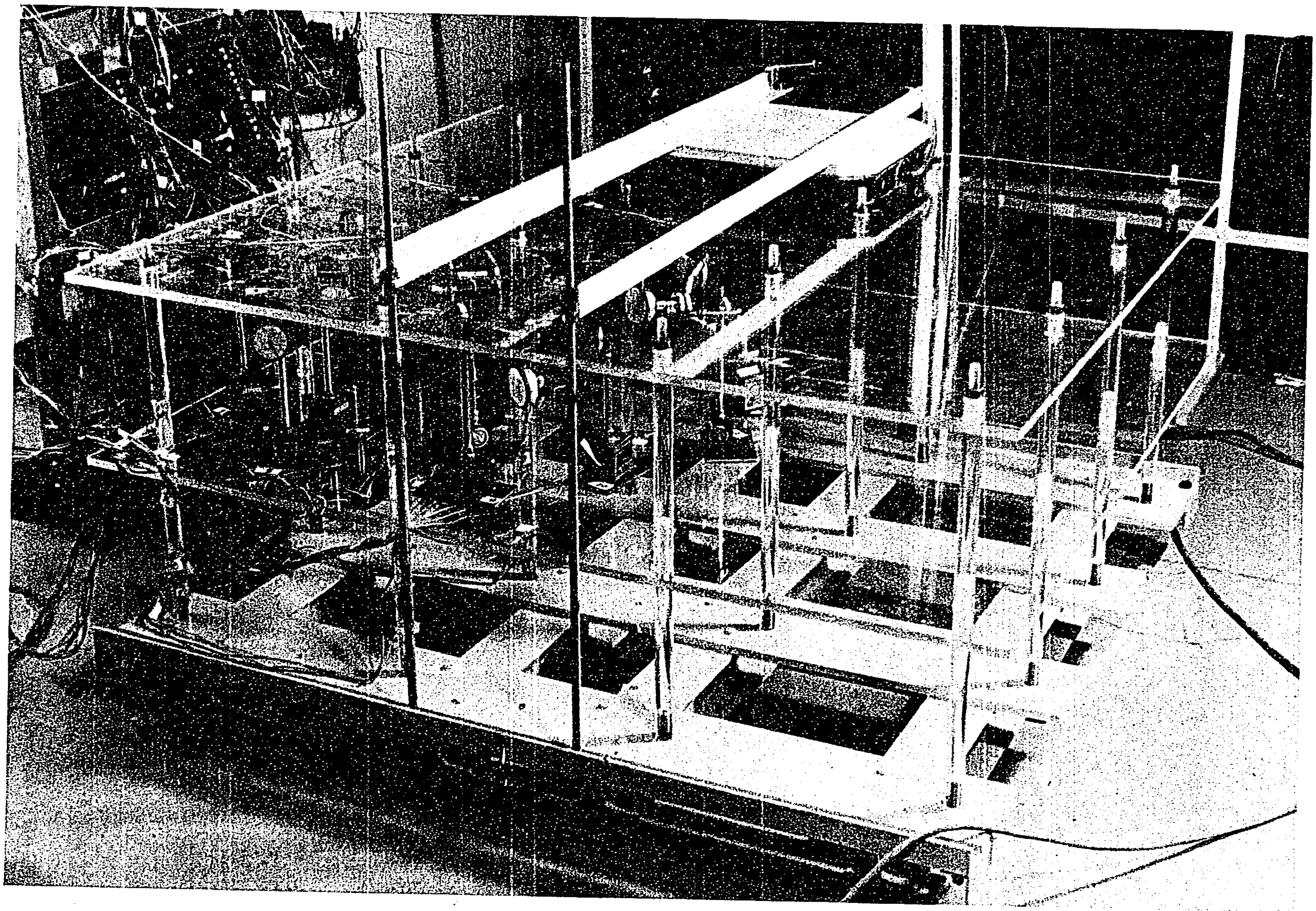


FIG. 4.9 THE MODEL AND THE INSTRUMENTS USED DURING THE TESTING OF THE MODEL

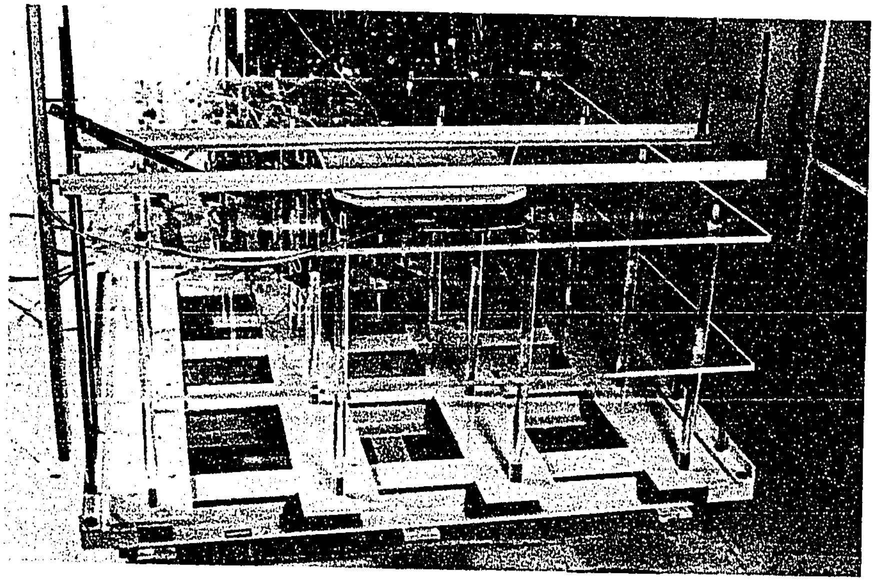


FIG. 4.10 LOADING SYSTEM

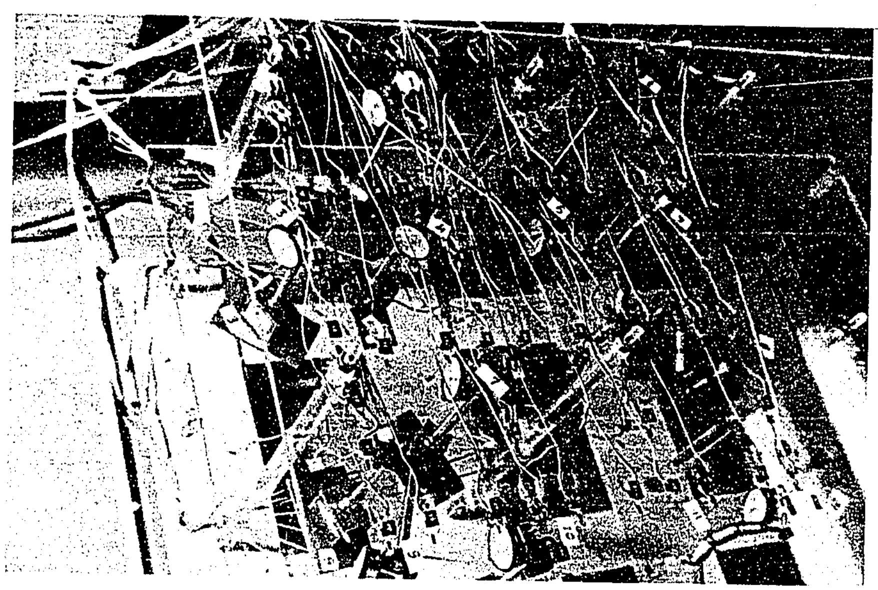


FIG. 4.11 ELECTRICAL STRAIN GAGES AND THE DEFLECTION DIAL GAGES

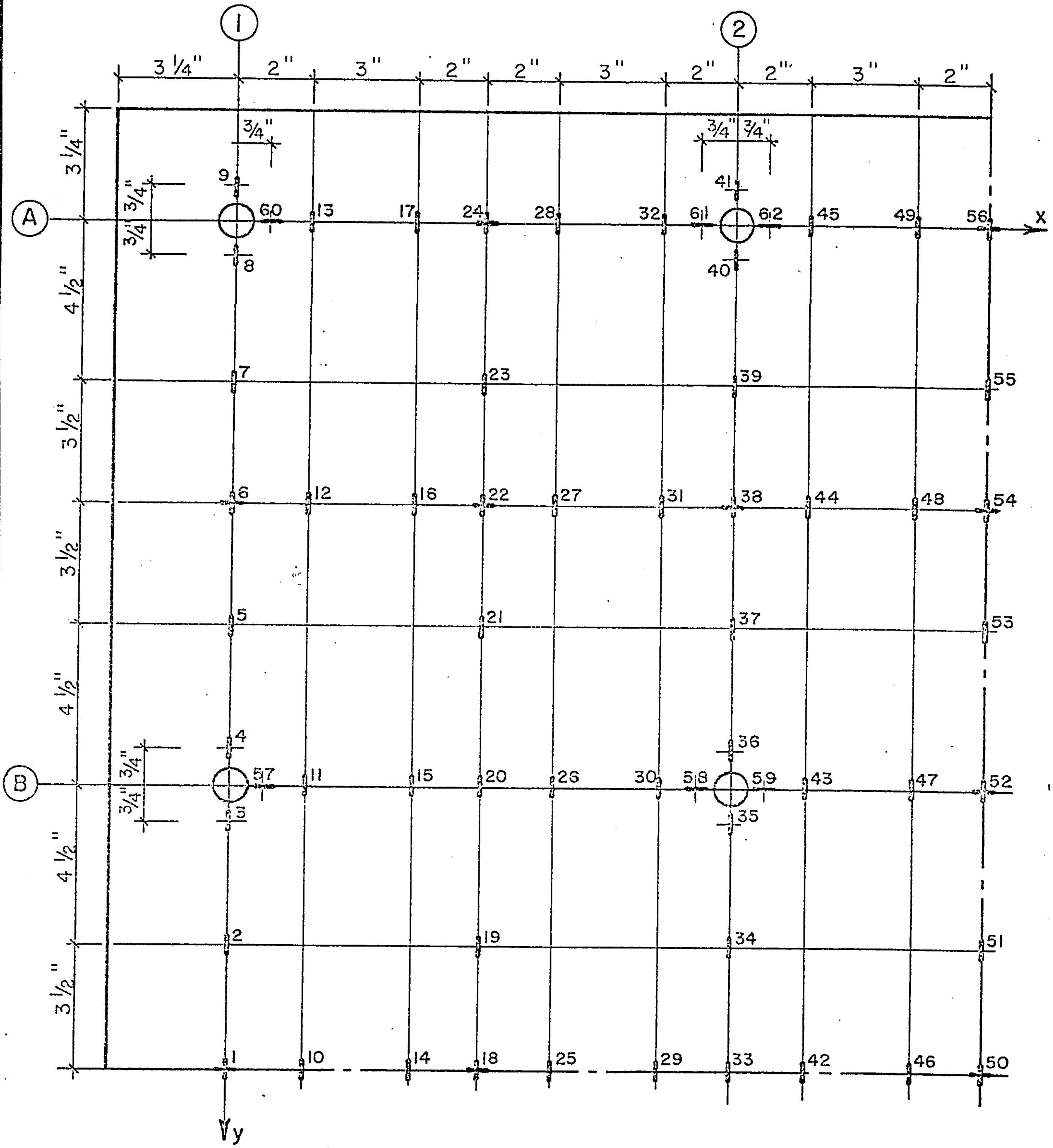


FIG. 4.12 : LOCATION AND DESIGNATION OF ELECTRICAL STRAIN GAGES ON BOTTOM OF PLATE (FIRST OR SECOND LEVEL)

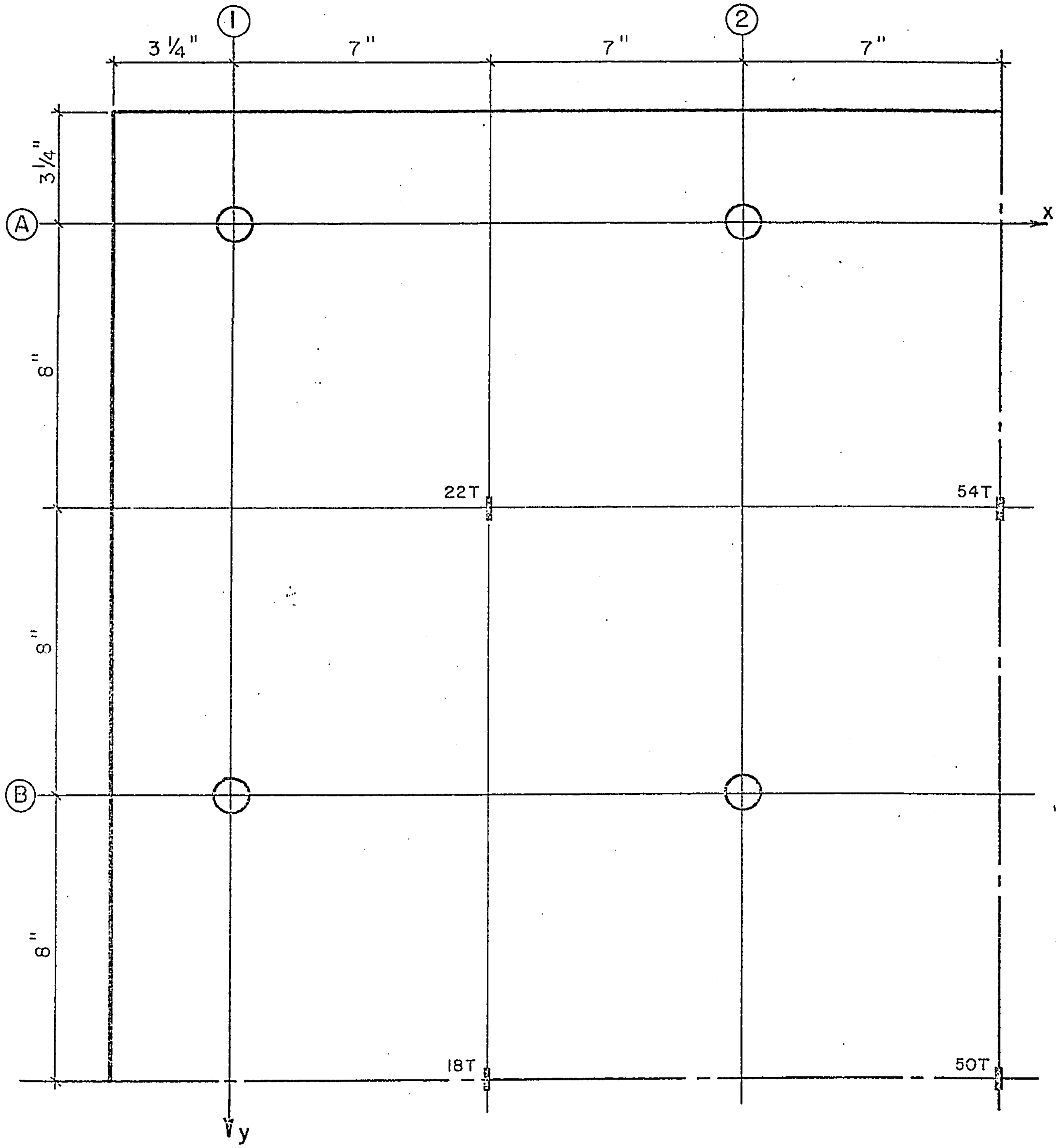


FIG. 4.13 : LOCATION AND DESIGNATION OF ELECTRICAL STRAIN GAGES ON TOP OF PLATE. (FIRST OR SECOND LEVEL)

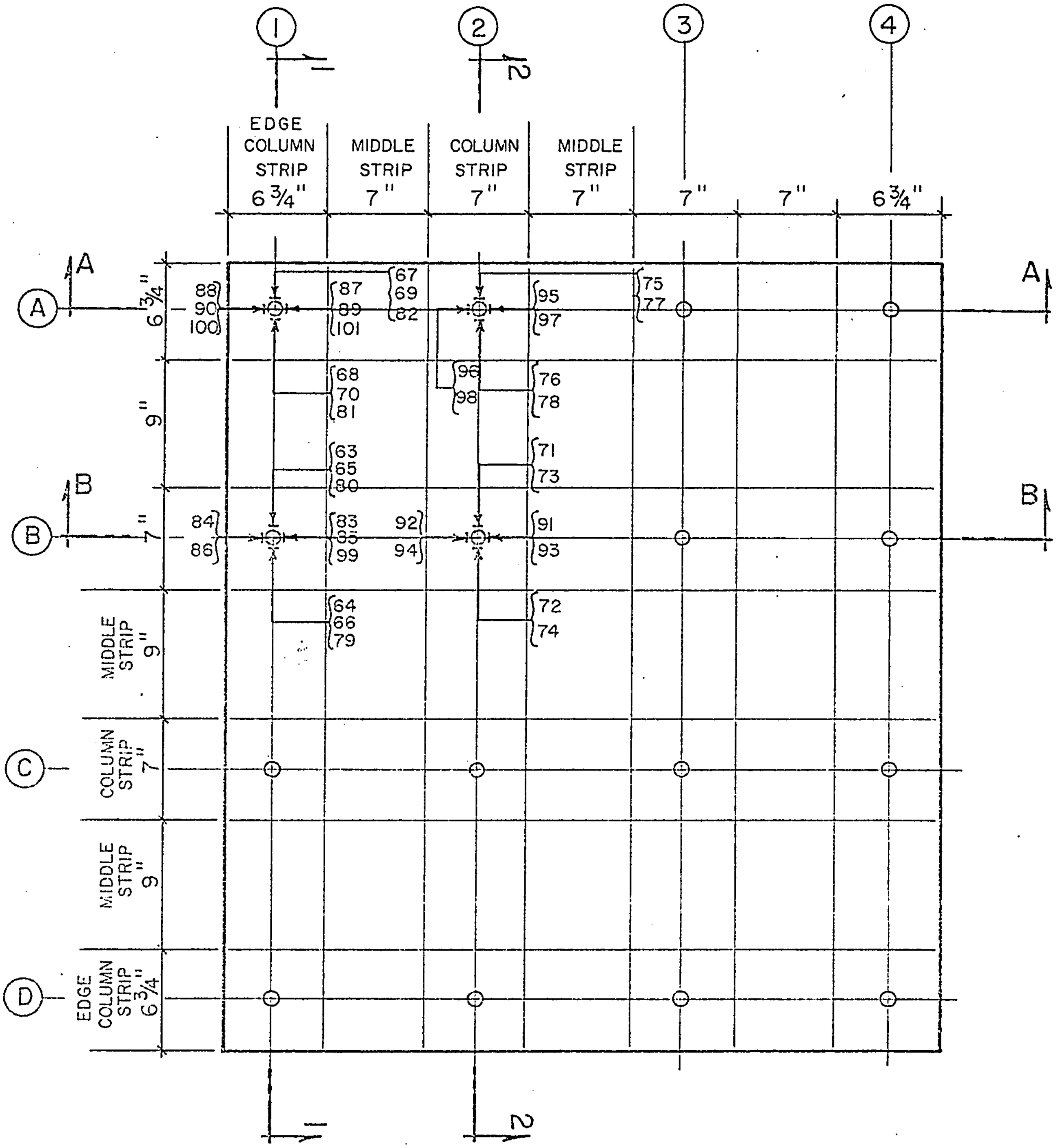


FIG. 4.14 : LOCATION AND DESIGNATION OF ELECTRICAL STRAIN GAGES ON COLUMNS.

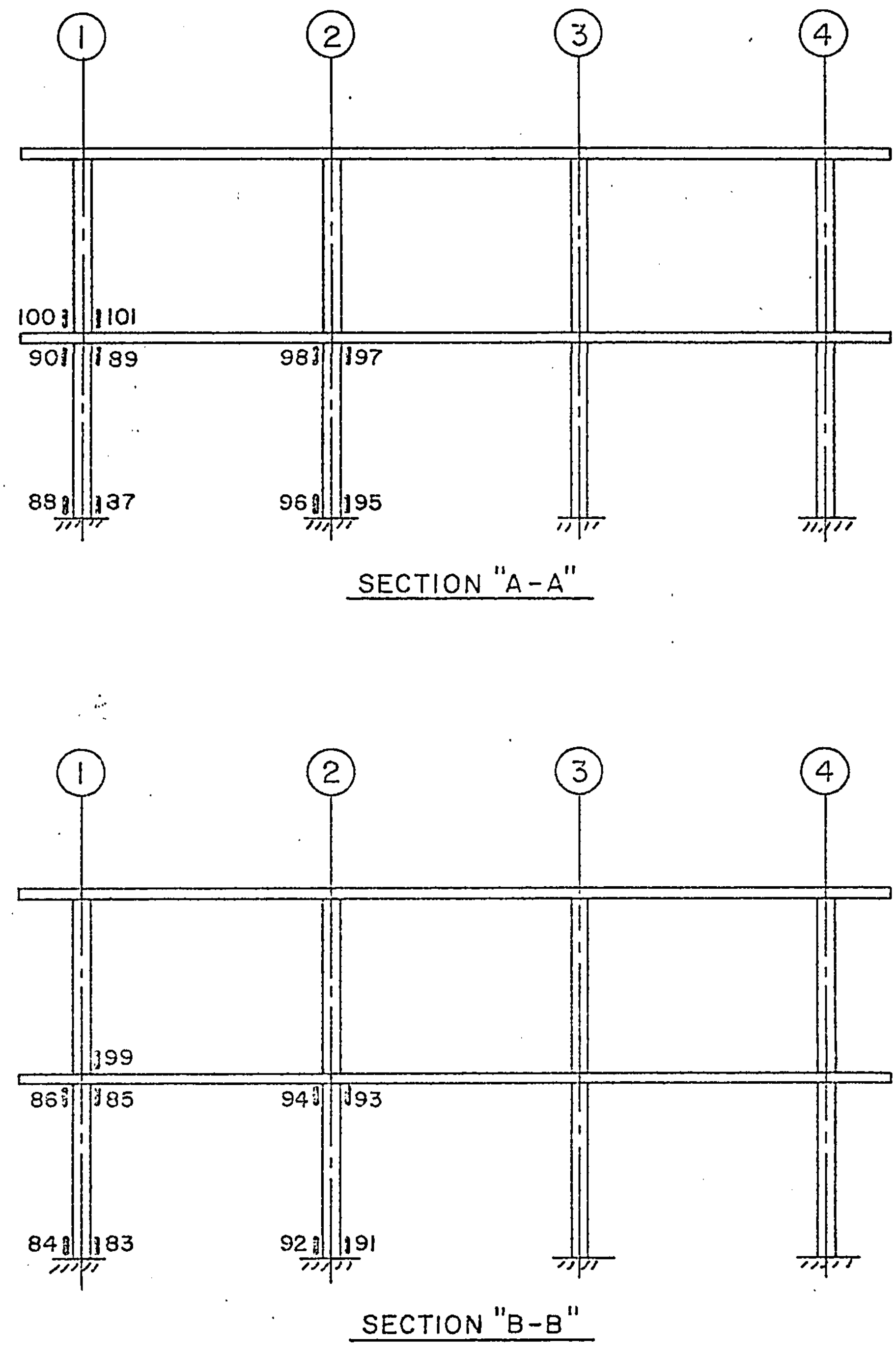
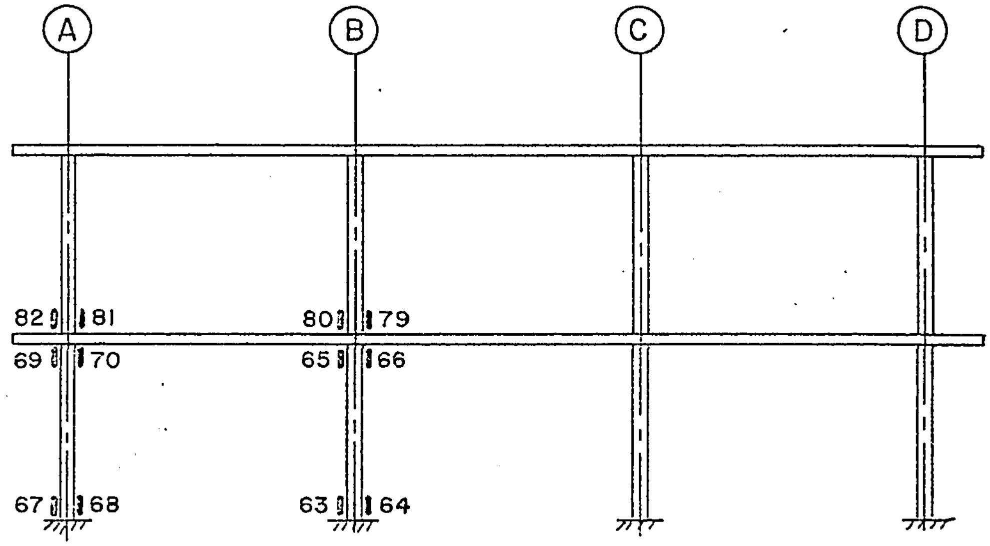
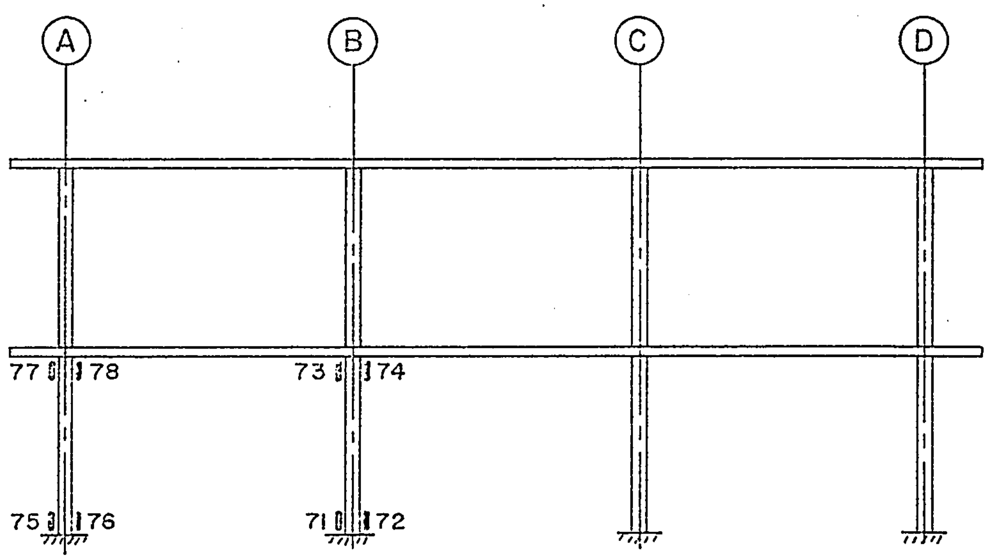


FIG. 4.15 : LOCATION AND DESIGNATION OF ELECTRICAL STRAIN GAGES ON COLUMNS.



SECTION "1-1"



SECTION "2-2"

FIG. 4.16: LOCATION AND DESIGNATION OF ELECTRICAL STRAIN GAGES ON COLUMNS.

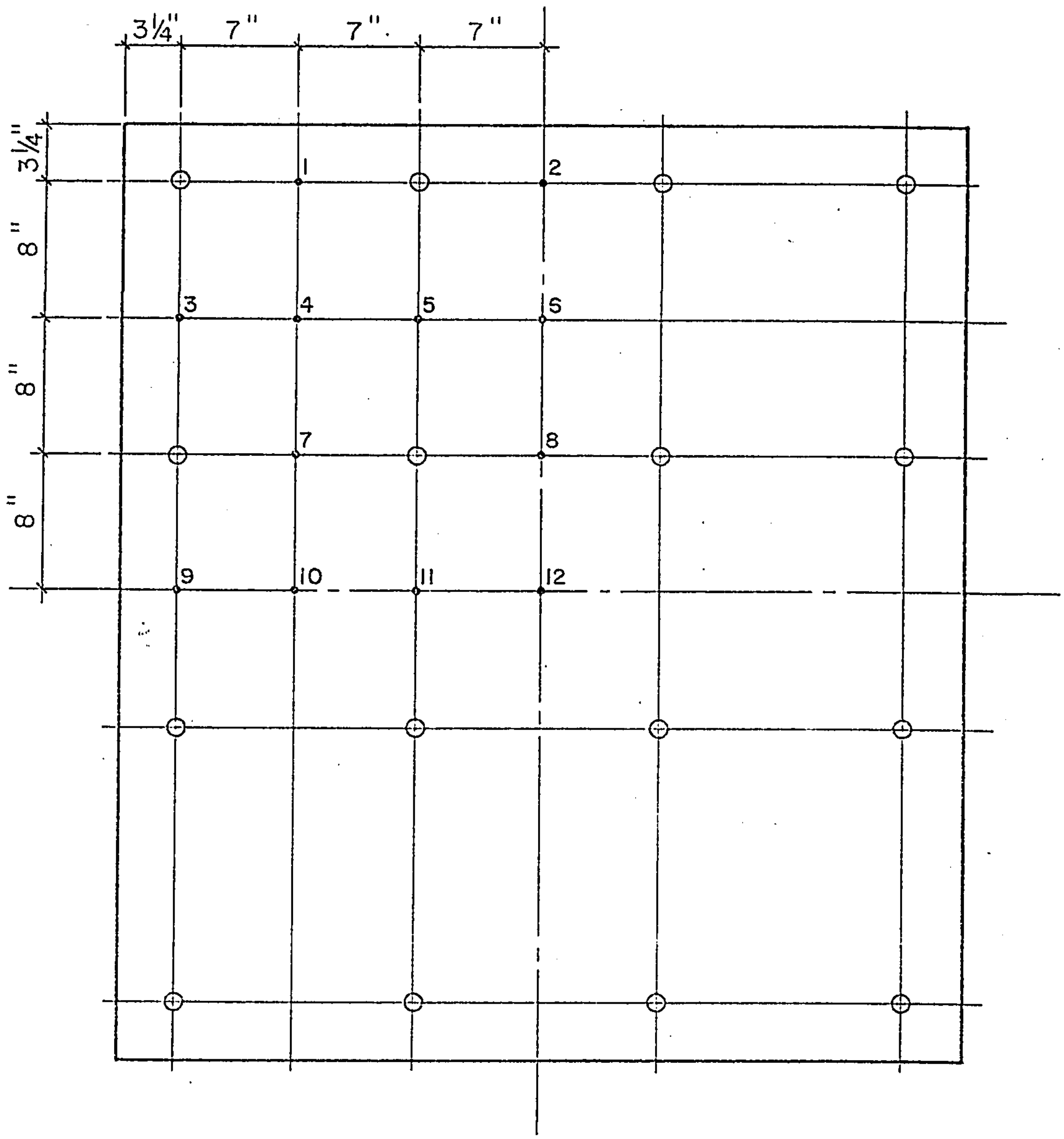


FIG. 4.17: LOCATION AND DESIGNATION OF DEFLECTION DIAL GAGES

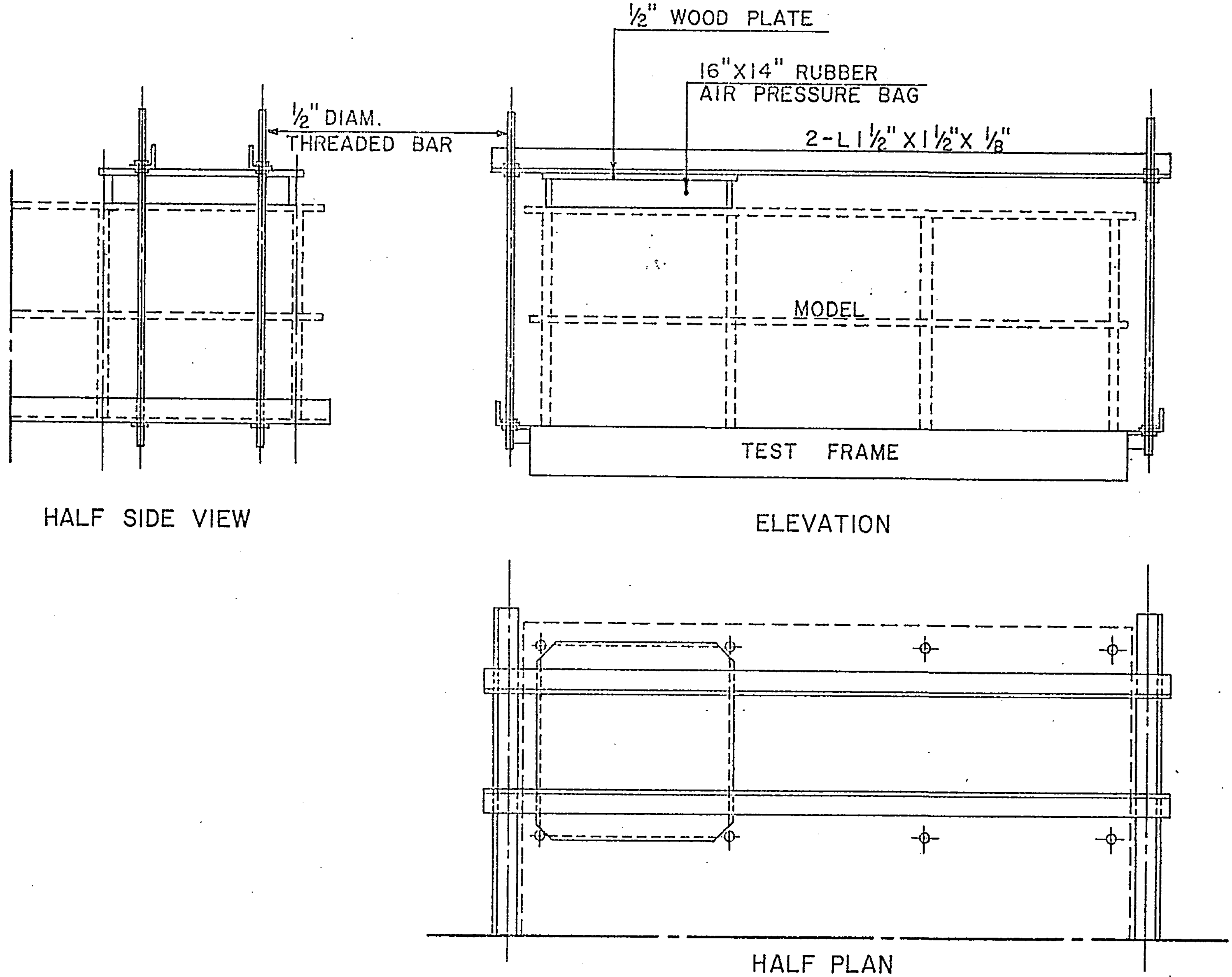
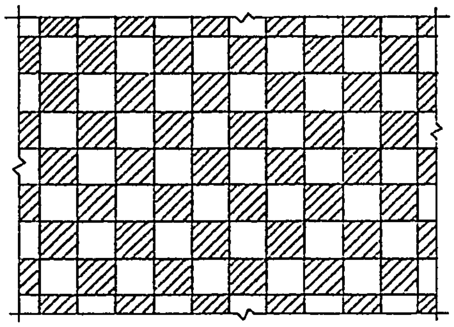
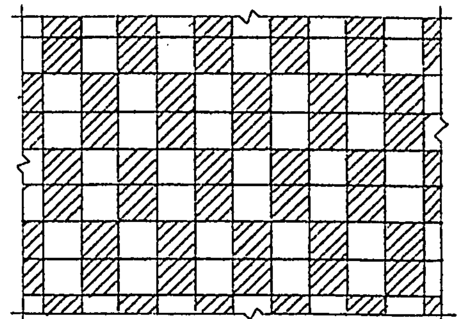


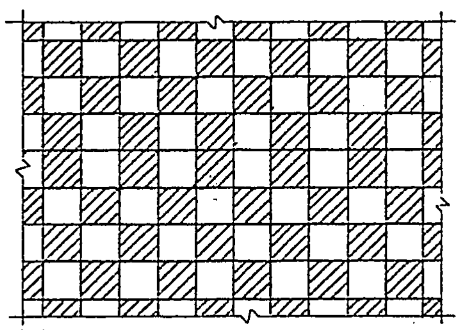
FIG. 4.18 : LOADING SYSTEM.



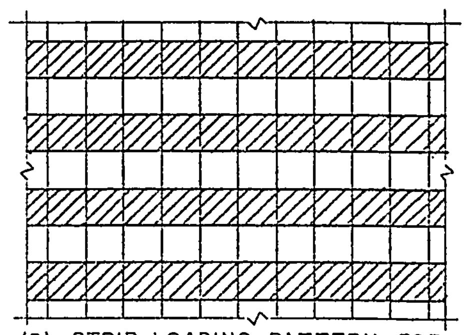
(A) CHECKER BOARD LOADING FOR MAXIMUM POSITIVE MOMENTS



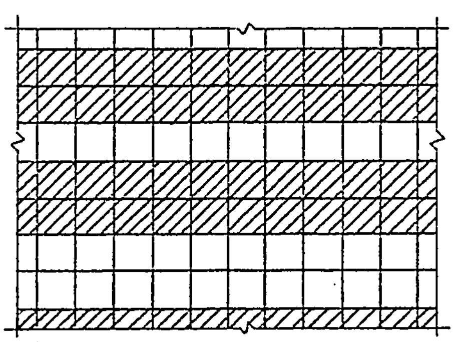
(B) CHECKER BOARD LOADING FOR MAXIMUM NEGATIVE MOMENTS



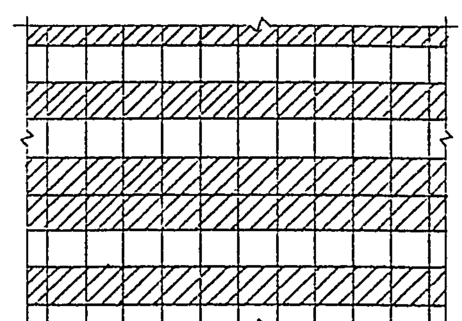
(C) CHECKER BOARD LOADING FOR MAXIMUM NEGATIVE MOMENTS.



(D) STRIP LOADING PATTERN FOR MAXIMUM POSITIVE MOMENTS

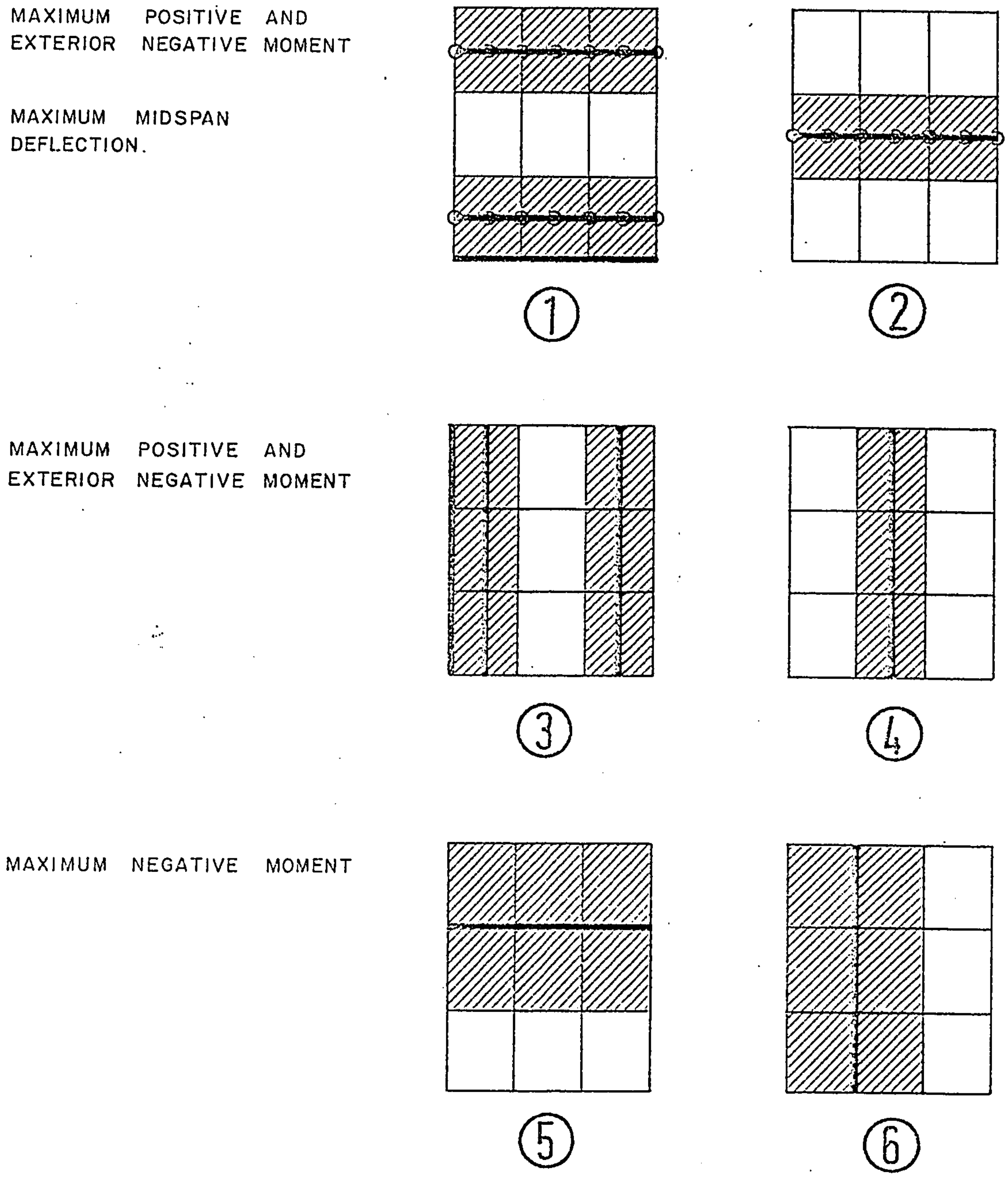


(E) STRIP LOADING PATTERN FOR MAXIMUM NEGATIVE MOMENTS



(F) STIP LOADING PATTERN FOR MAXIMUM NEGATIVE MOMENTS

FIG. 5.1 PARTIAL LOADING PATTERNS FOR MAXIMUM POSITIVE AND NEGATIVE MOMENTS.



━━ INDICATE SECTIONS OF MAXIMIZED MOMENT

o INDICATE LOCATIONS OF MAXIMIZED DEFLECTION

FIG. 5.2 : LOADING PATTERNS

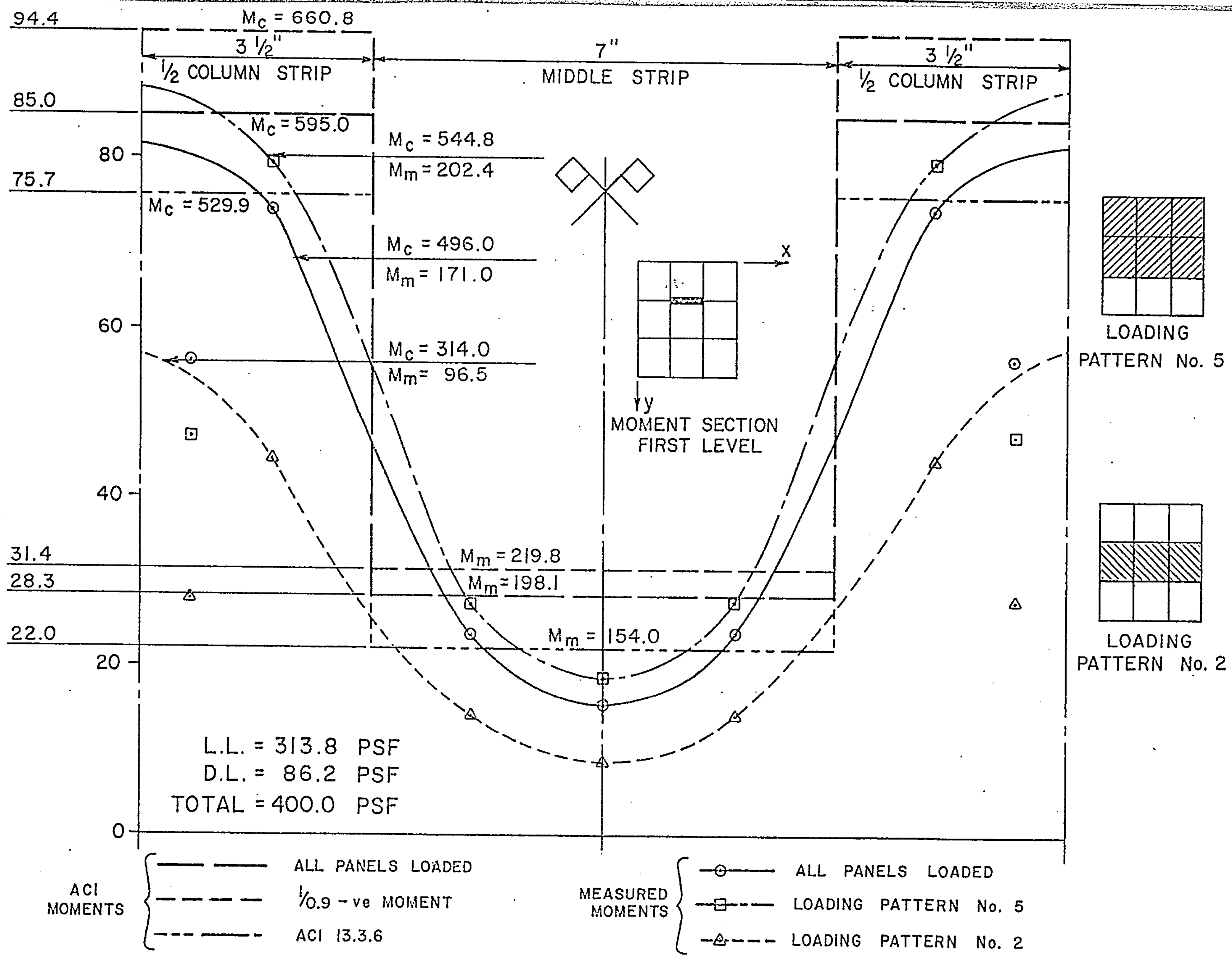


FIG. 5.3: COMPARISON OF ACI MOMENTS AND MEASURED MOMENTS FOR AN INTERIOR SPAN (MOMENT VARIATION ACROSS WIDTH OF MAXIMUM NEGATIVE MOMENT SECTION)

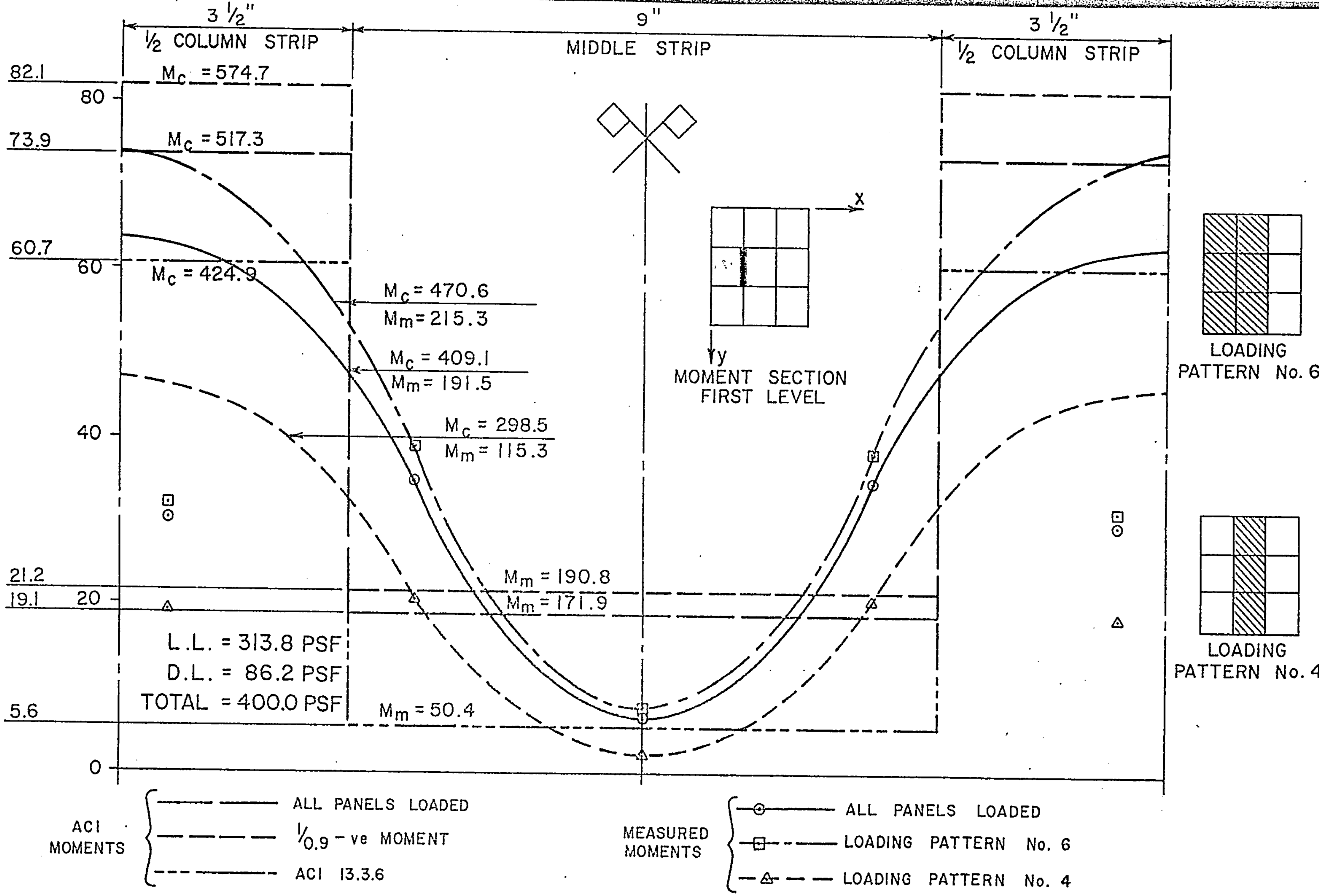


FIG. 5.4 : COMPARISON OF ACI MOMENTS AND MEASURED MOMENTS FOR AN INTERIOR SPAN
(MOMENT VARIATION ACROSS WIDTH OF MAXIMUM NEGATIVE MOMENT SECTION)

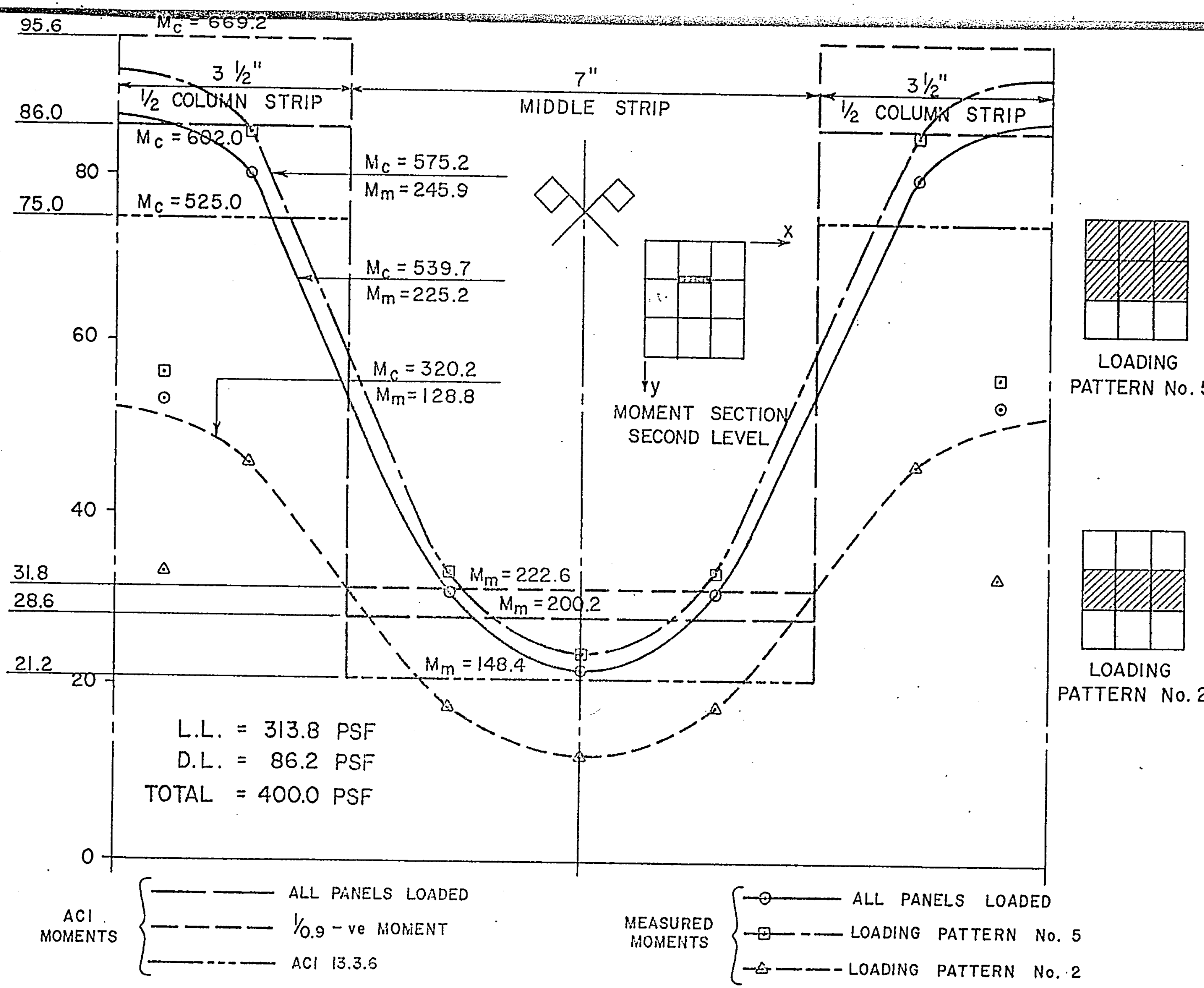


FIG. 5.5 : COMPARISON OF ACI MOMENTS AND MEASURED MOMENTS FOR AN INTERIOR SPAN (MOMENT VARIATION ACROSS WIDTH OF MAXIMUM NEGATIVE MOMENT SECTION)

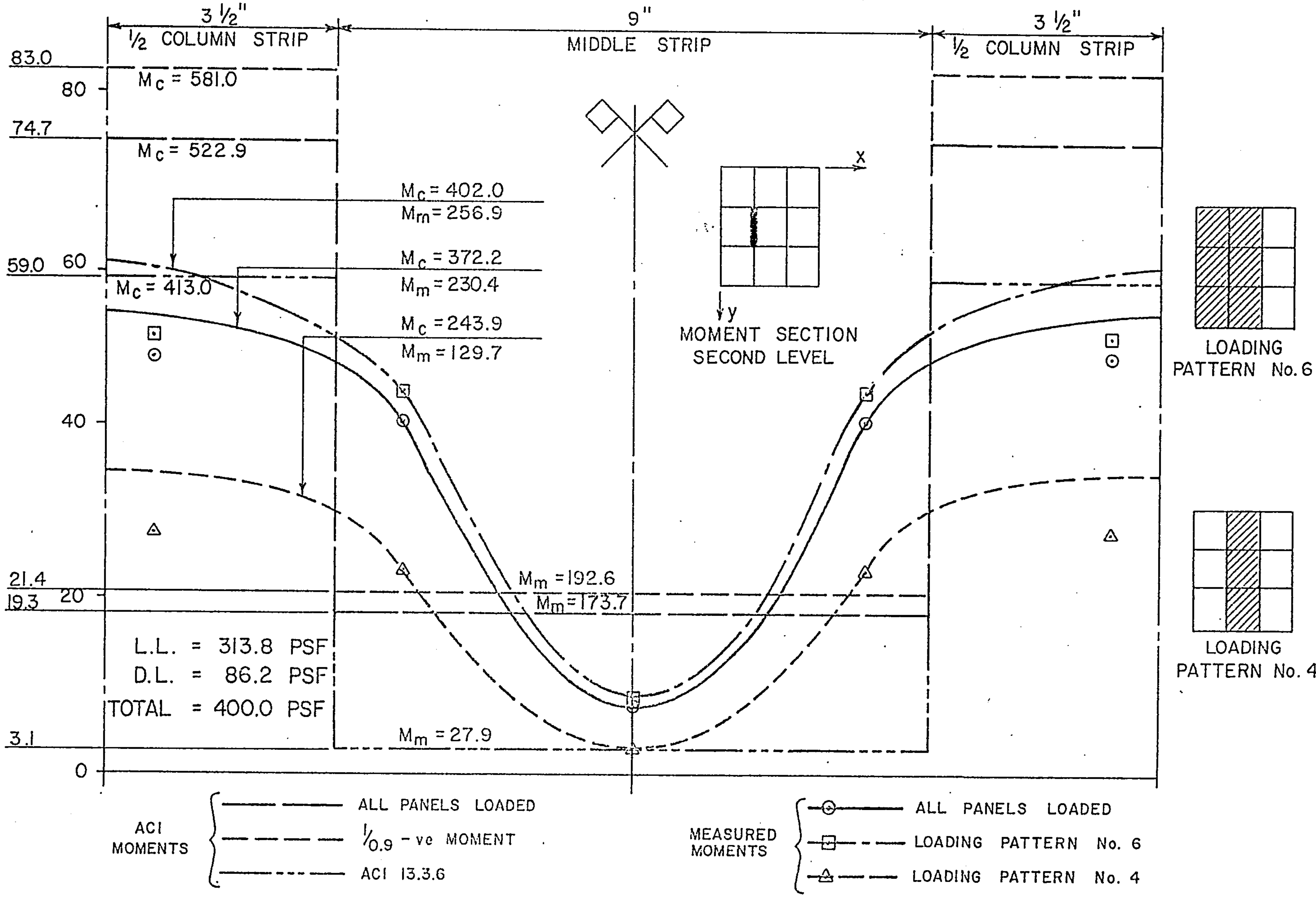


FIG. 5.6 : COMPARISON OF ACI MOMENTS AND MEASURED MOMENTS FOR AN INTERIOR SPAN (MOMENT VARIATION ACROSS WIDTH OF MAXIMUM NEGATIVE MOMENT SECTION)

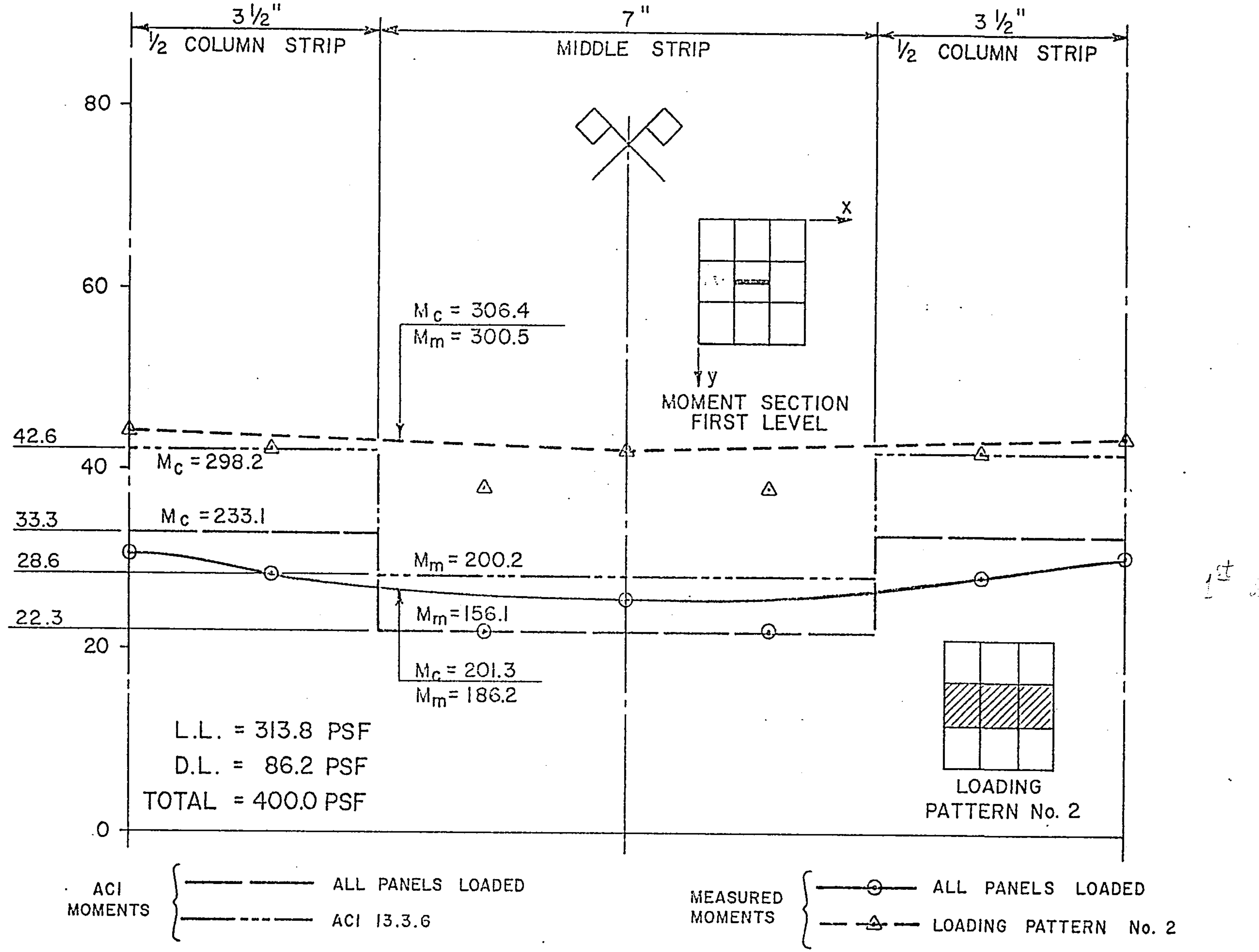


FIG. 5.7 : COMPARISON OF ACI MOMENTS AND MEASURED MOMENTS FOR AN INTERIOR SPAN (MOMENT VARIATION ACROSS WIDTH OF MAXIMUM POSITIVE MOMENT SECTION)

1st level

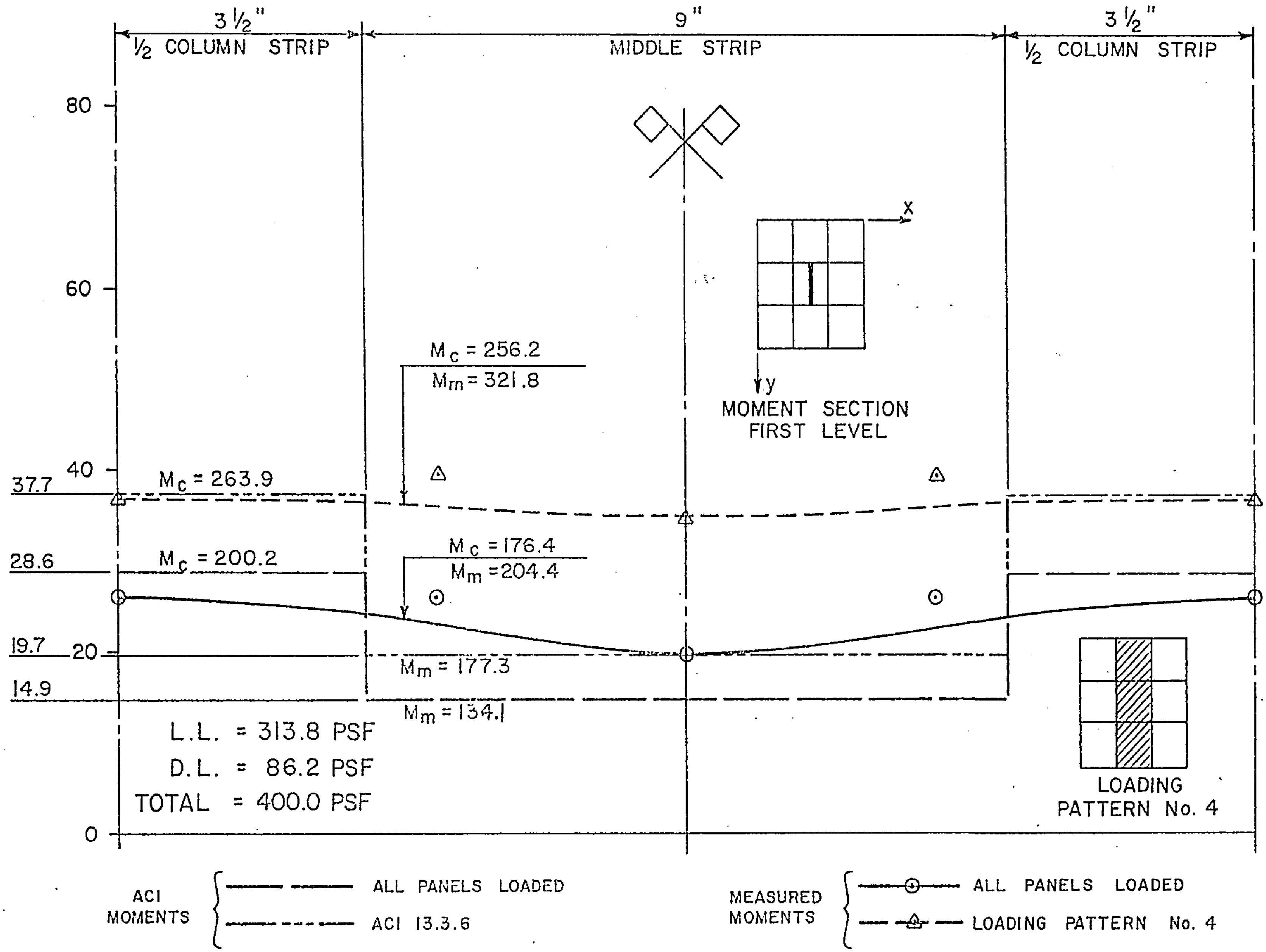


FIG. 5.8 : COMPARISON OF ACI MOMENTS AND MEASURED MOMENTS FOR AN INTERIOR SPAN
(MOMENT VARIATION ACROSS WIDTH OF MAXIMUM POSITIVE MOMENT SECTION)

1st level

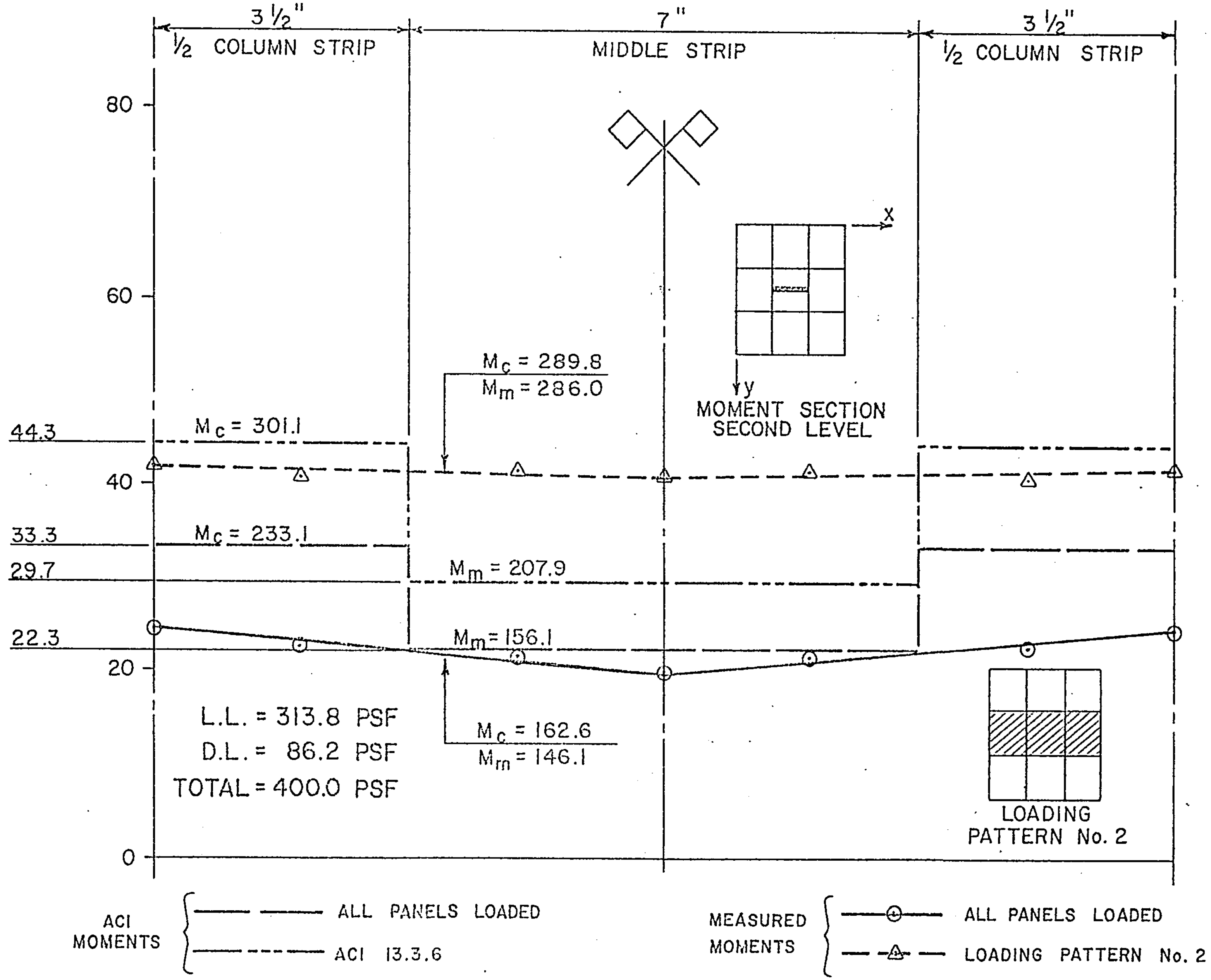


FIG. 5.9 : COMPARISON OF ACI MOMENTS AND MEASURED MOMENTS FOR AN INTERIOR SPAN (MOMENT VARIATION ACROSS WIDTH OF MAXIMUM POSITIVE MOMENT SECTION)

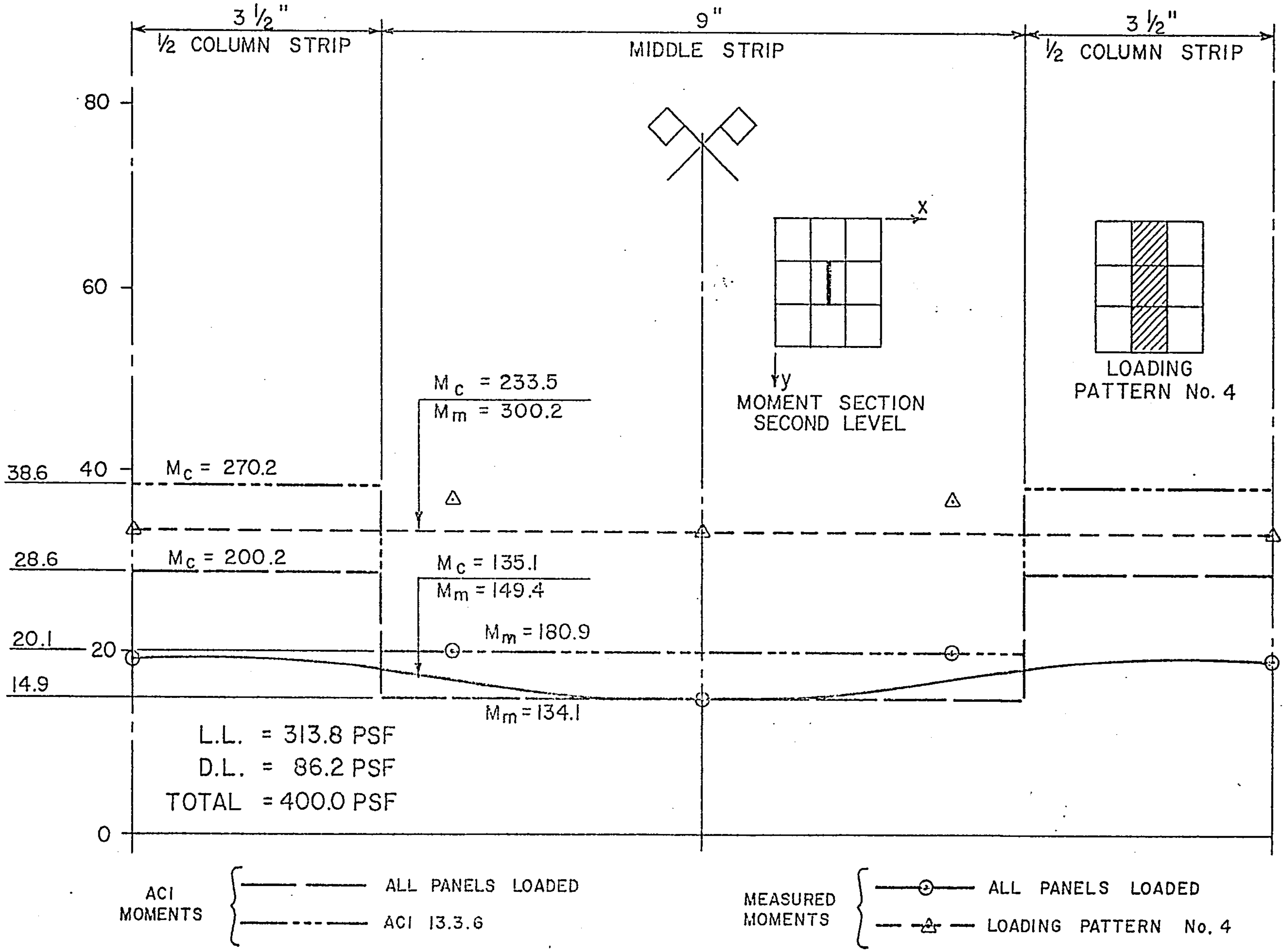


FIG. 5.10: COMPARISON OF ACI MOMENTS AND MEASURED MOMENTS FOR AN INTERIOR SPAN (MOMENT VARIATION ACROSS WIDTH OF MAXIMUM POSITIVE MOMENT SECTION)

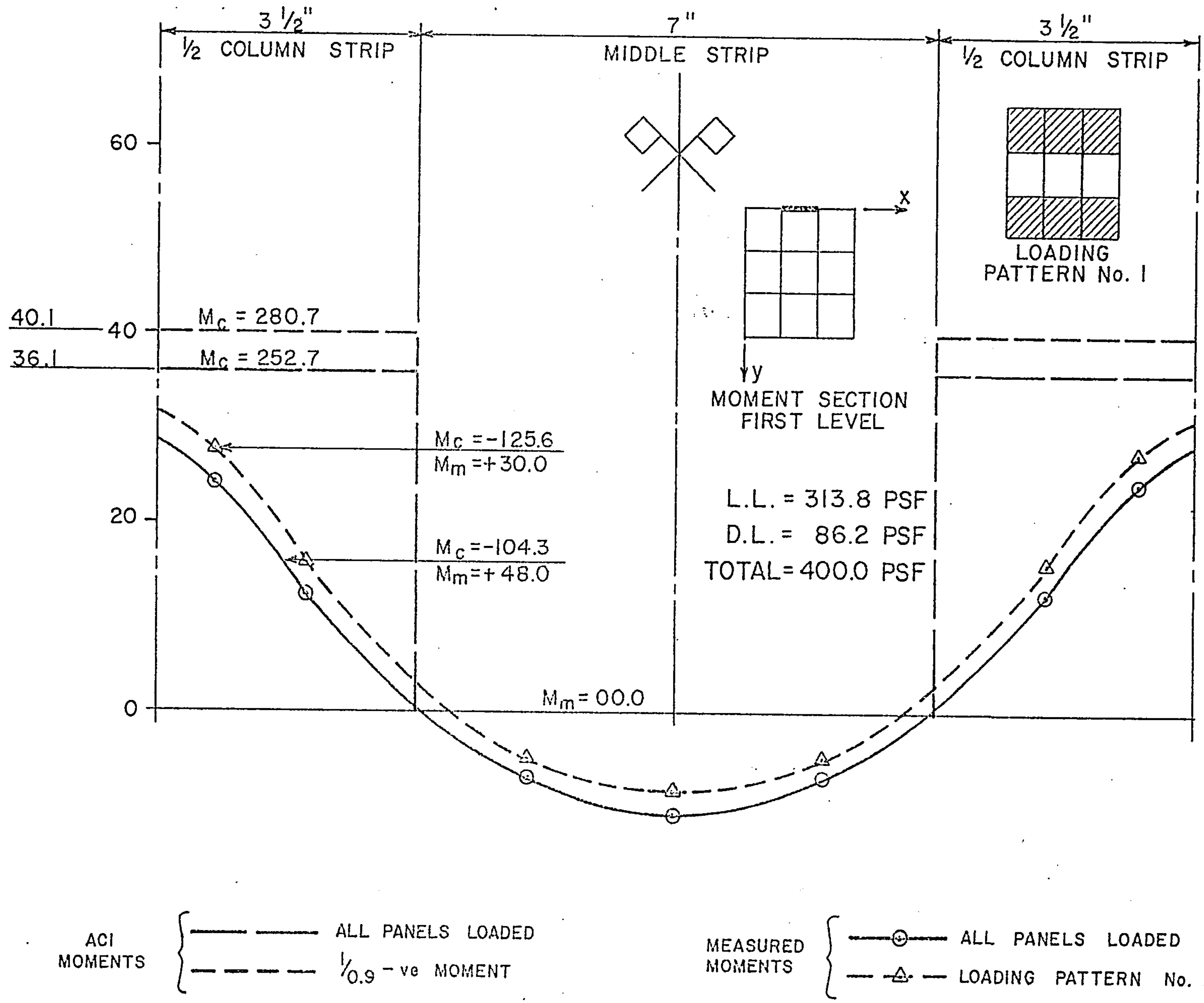


FIG. 5.11 : COMPARISON OF ACI MOMENTS AND MEASURED MOMENTS FOR AN END SPAN (MOMENT VARIATION ACROSS WIDTH OF MAXIMUM NEGATIVE MOMENT SECTION)

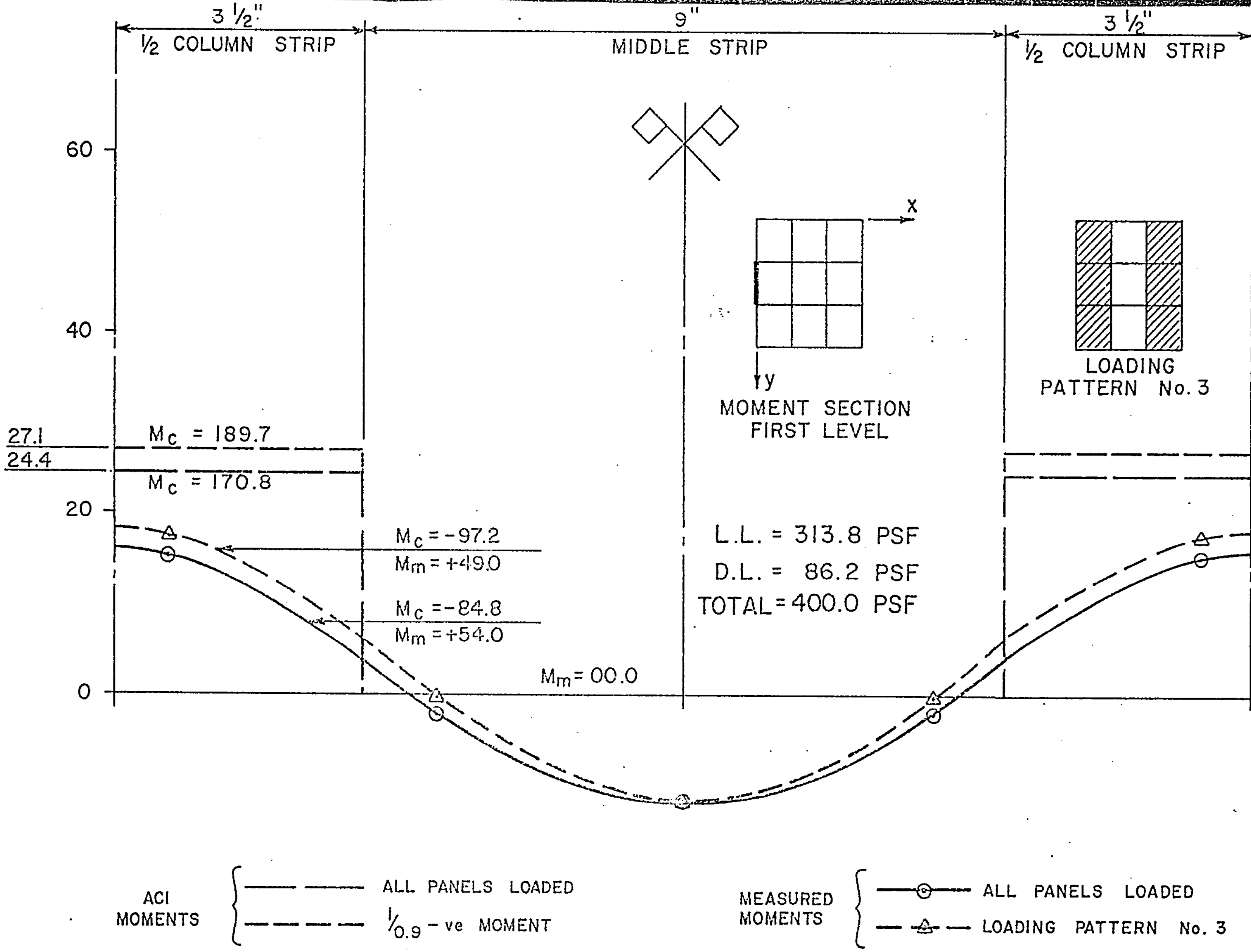


FIG. 5.12 : COMPARISON OF ACI MOMENTS AND MEASURED MOMENTS FOR AN END SPAN (MOMENT VARIATION ACROSS WIDTH OF MAXIMUM NEGATIVE MOMENT SECTION)

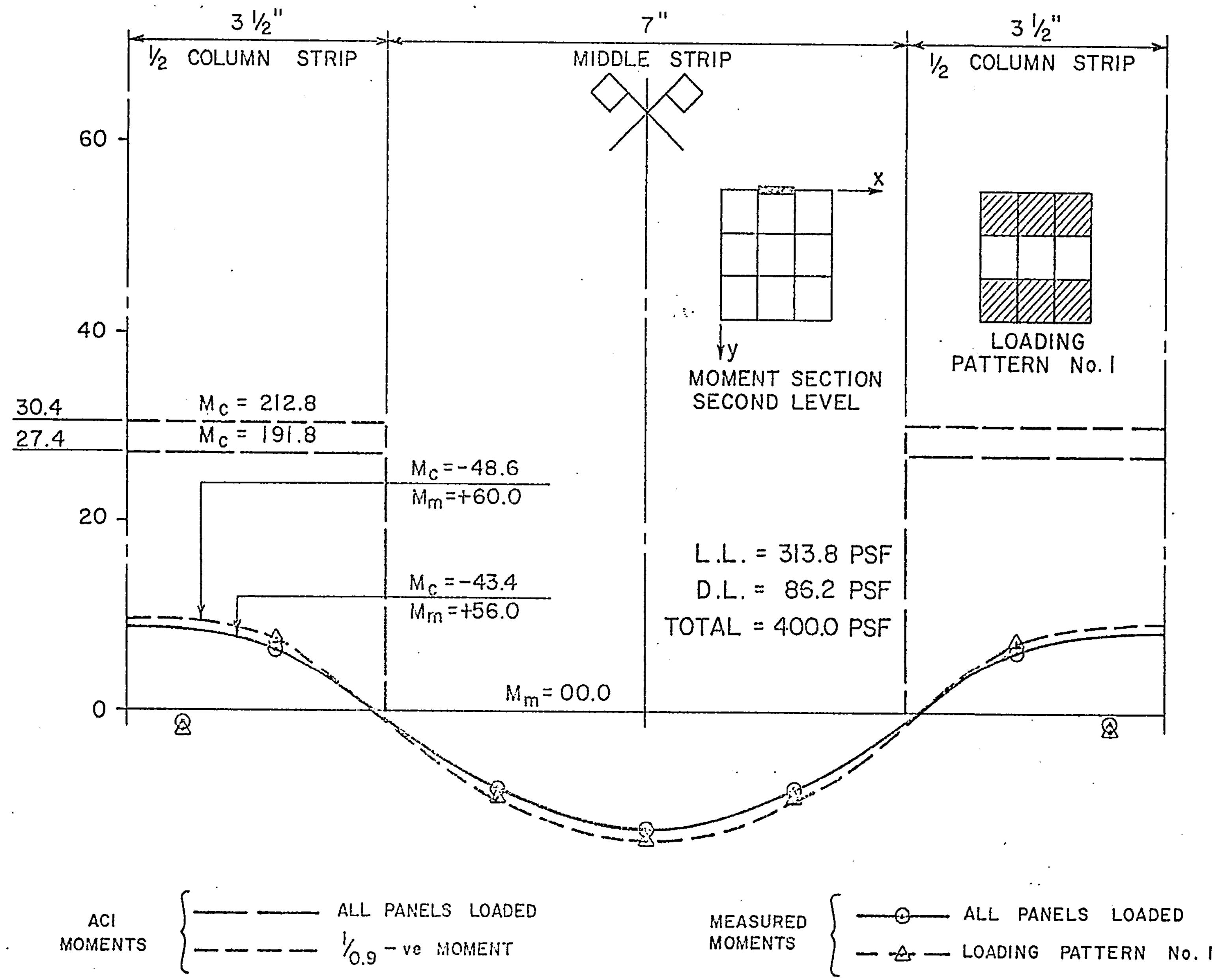


FIG. 5.13 : COMPARISON OF ACI MOMENTS AND MEASURED MOMENTS FOR AN END SPAN (MOMENT VARIATION ACROSS WIDTH OF MAXIMUM NEGATIVE MOMENT SECTION)

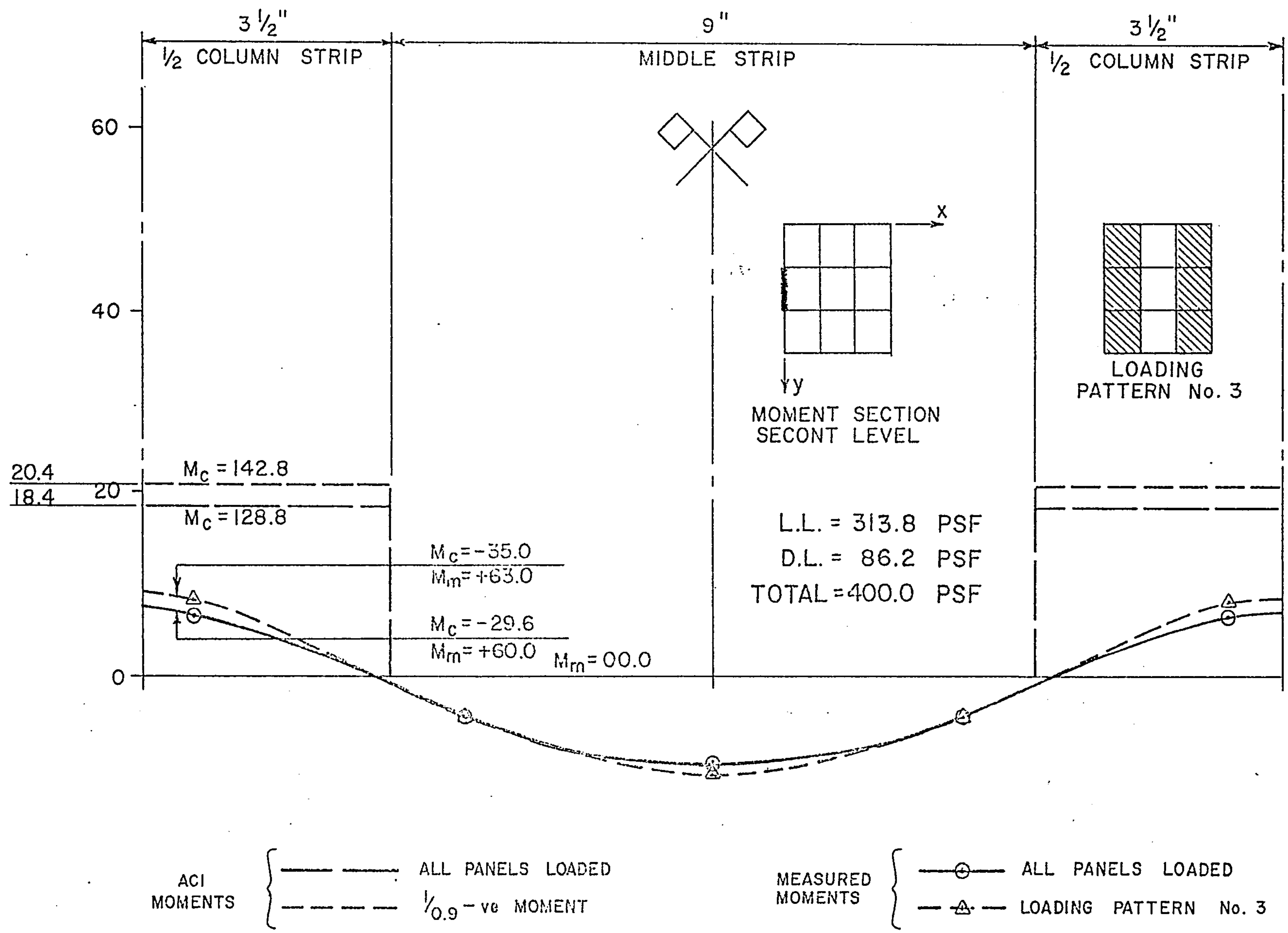


FIG. 5.14 : COMPARISON OF ACI MOMENTS AND MEASURED MOMENTS FOR AN END SPAN (MOMENT VARIATION ACROSS WIDTH OF MAXIMUM NEGATIVE MOMENT SECTION)

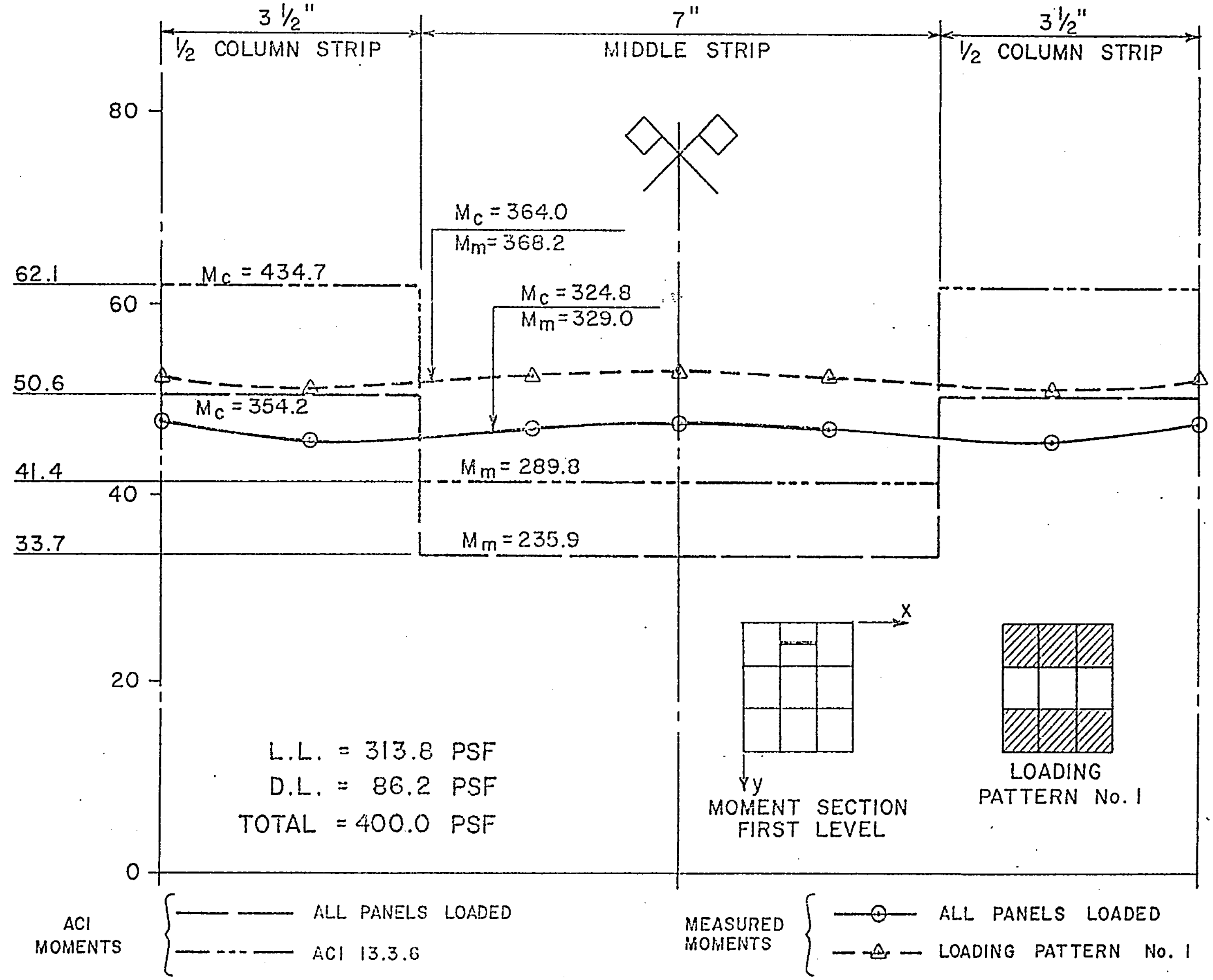


FIG. 5.15 : COMPARISON OF ACI MOMENTS AND MEASURED MOMENTS FOR AN END SPAN
(MOMENT VARIATION ACROSS WIDTH OF MAXIMUM POSITIVE MOMENT SECTION)

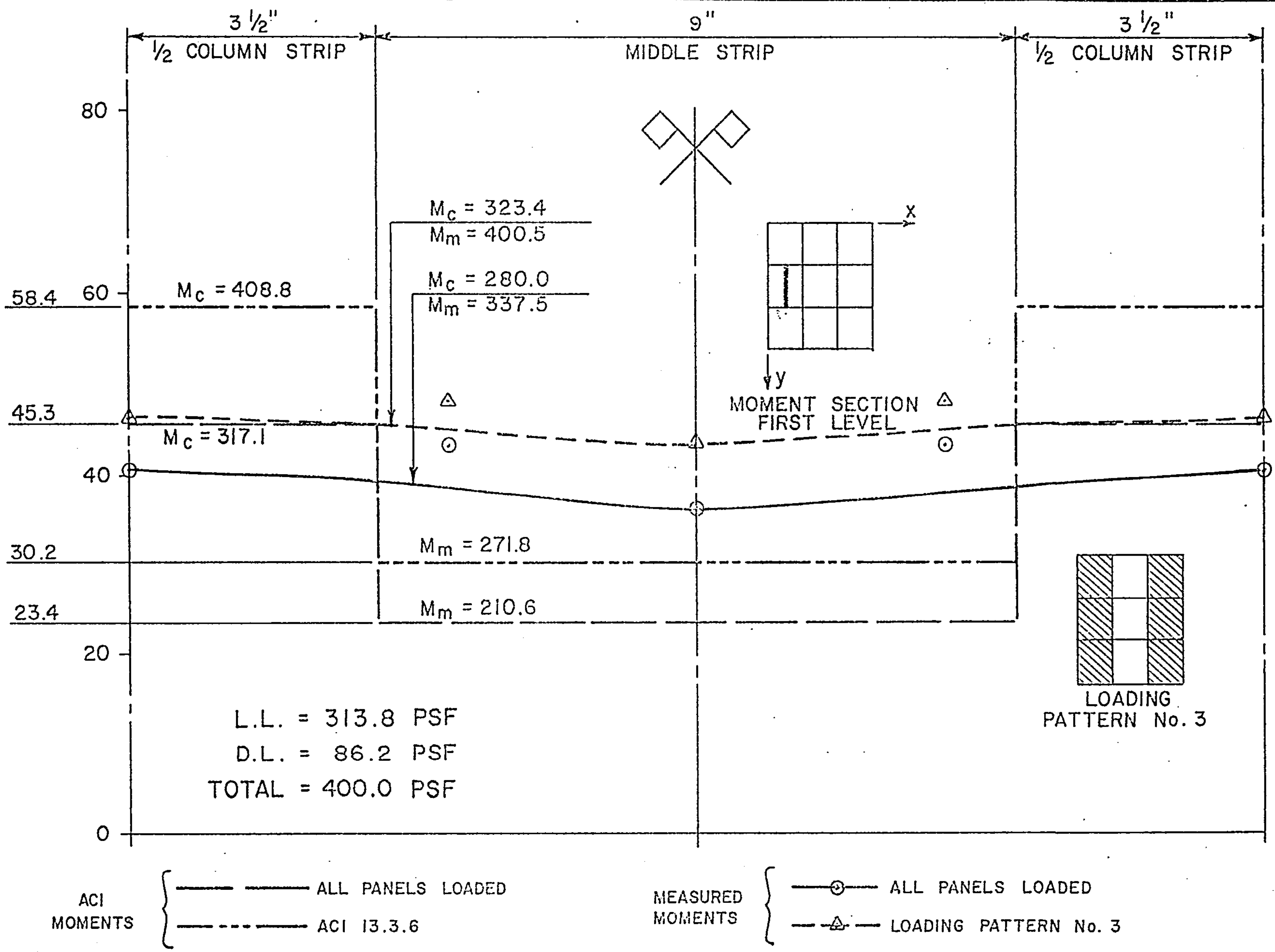


FIG. 5.16: COMPARISON OF ACI MOMENTS AND MEASURED MOMENTS FOR AN END SPAN (MOMENT VARIATION ACROSS WIDTH OF MAXIMUM POSITIVE MOMENT SECTION)

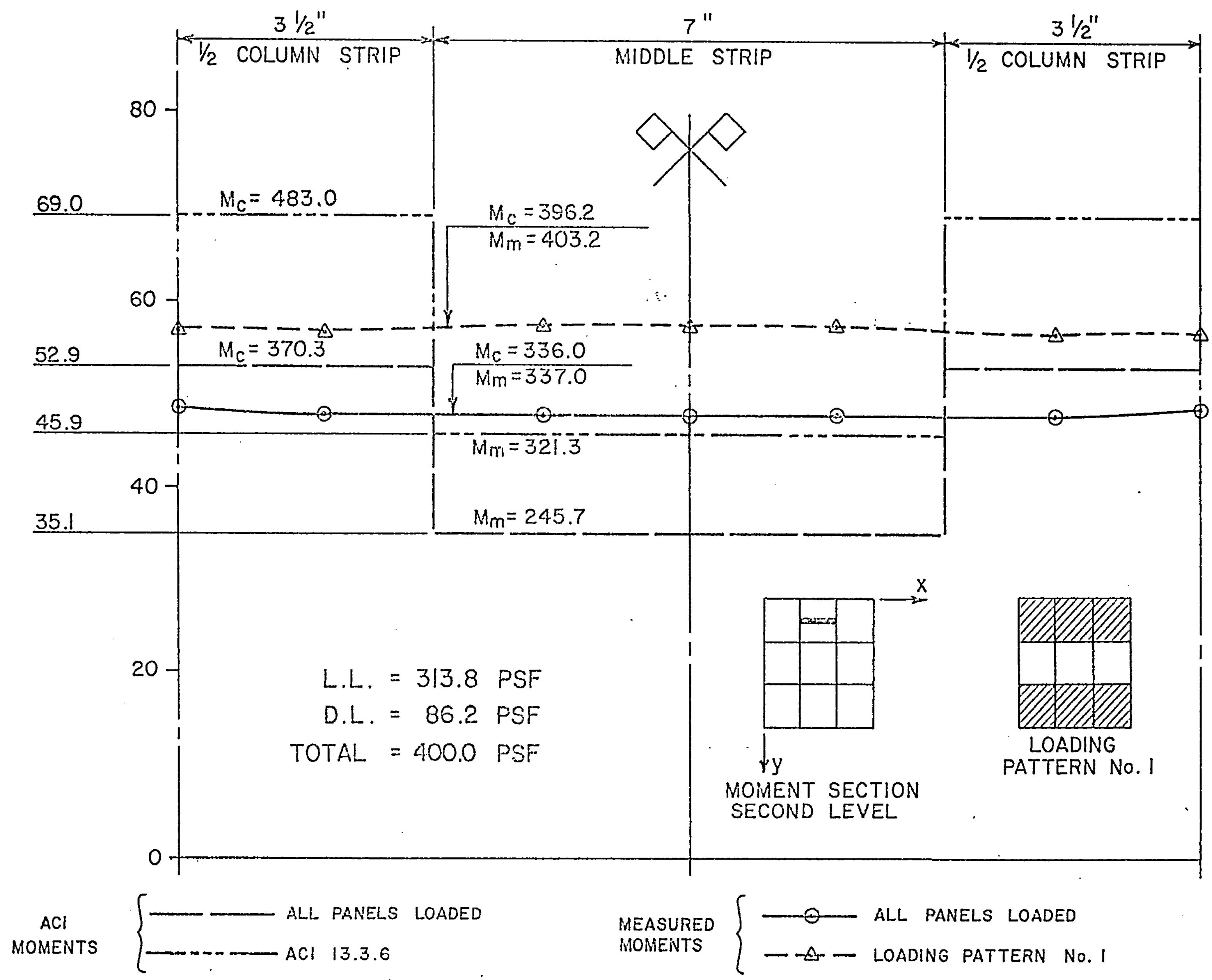


FIG. 5.17 : COMPARISON OF ACI MOMENTS AND MEASURED MOMENTS FOR AN END SPAN (MOMENT VARIATION ACROSS WIDTH OF MAXIMUM POSITIVE MOMENT SECTION)

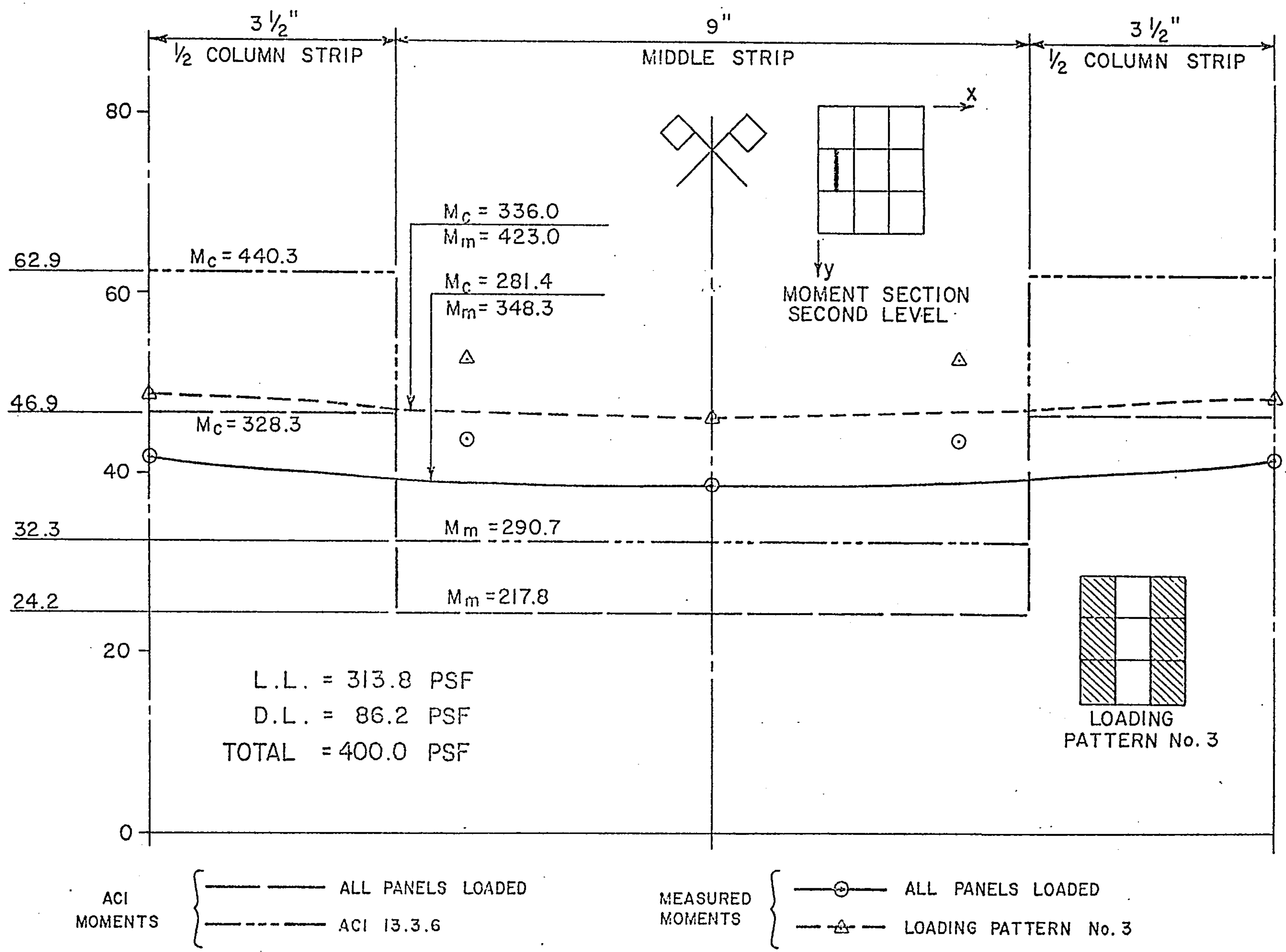


FIG. 5.18 : COMPARISON OF ACI MOMENTS AND MEASURED MOMENTS FOR AN END SPAN
(MOMENT VARIATION ACROSS WIDTH OF MAXIMUM POSITIVE MOMENT SECTION)

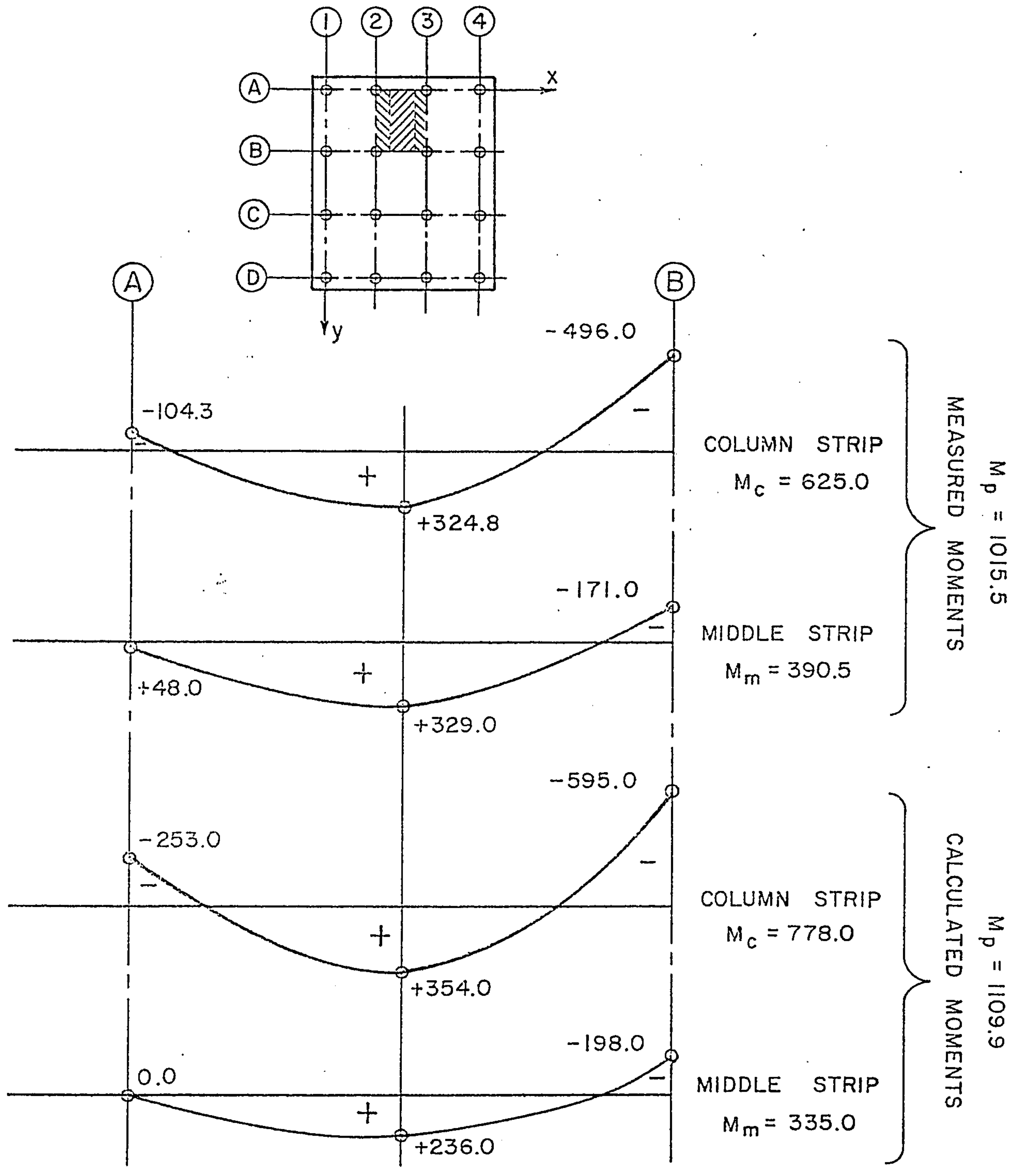


FIG. 5.19: COMPARISON OF CALCULATED MOMENTS AND MEASURED MOMENTS, FOR AN END PANEL IN Y-DIRECTION. FIRST LEVEL.

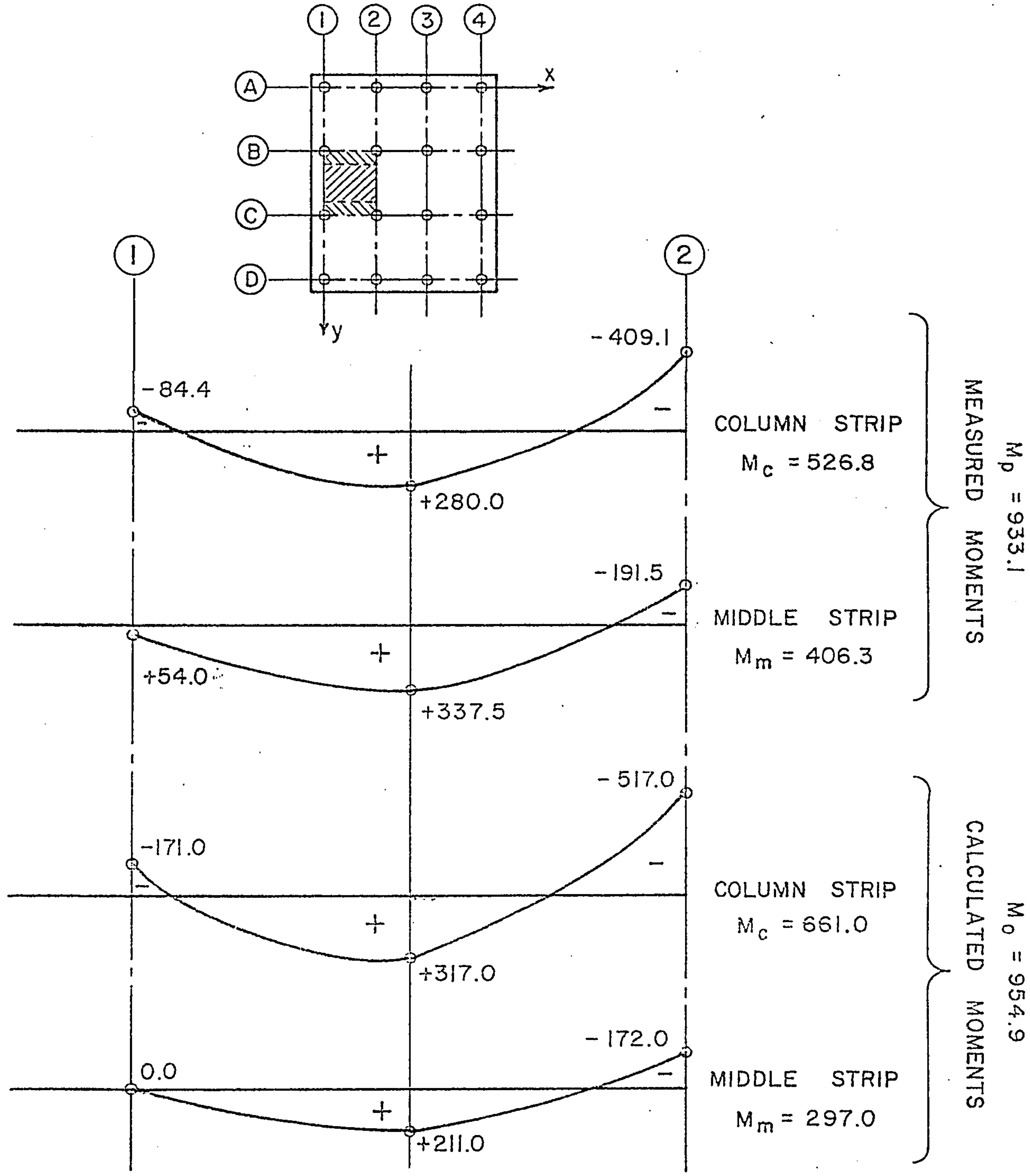


FIG. 5.20: COMPARISON OF CALCULATED MOMENTS AND MEASURED MOMENTS, FOR AN END PANEL IN X-DIRECTION. FIRST LEVEL.

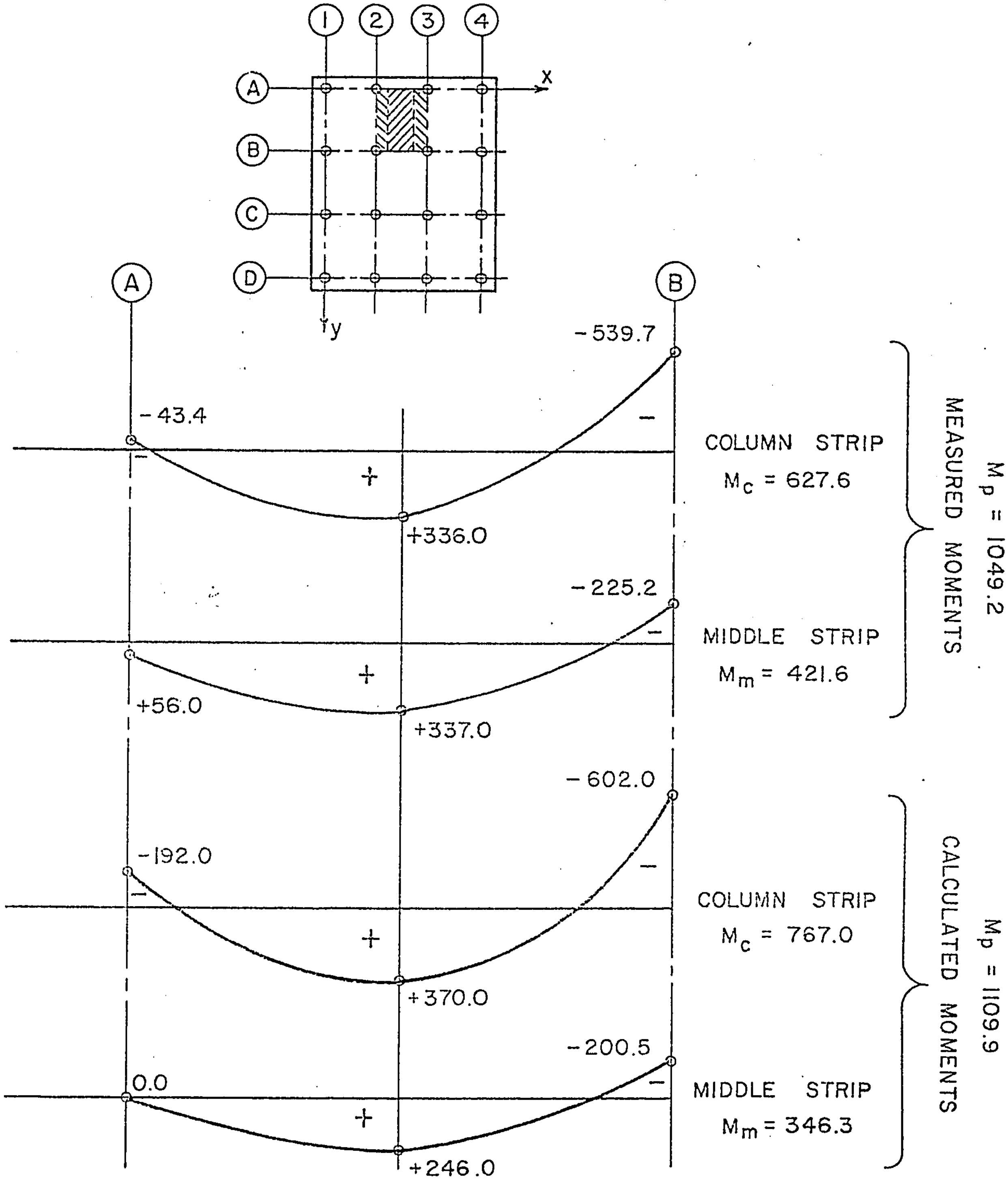


FIG. 5.21: COMPARISON OF CALCULATED MOMENTS AND MEASURED MOMENTS, FOR AN END PANEL IN Y-DIRECTION. SECOND LEVEL.

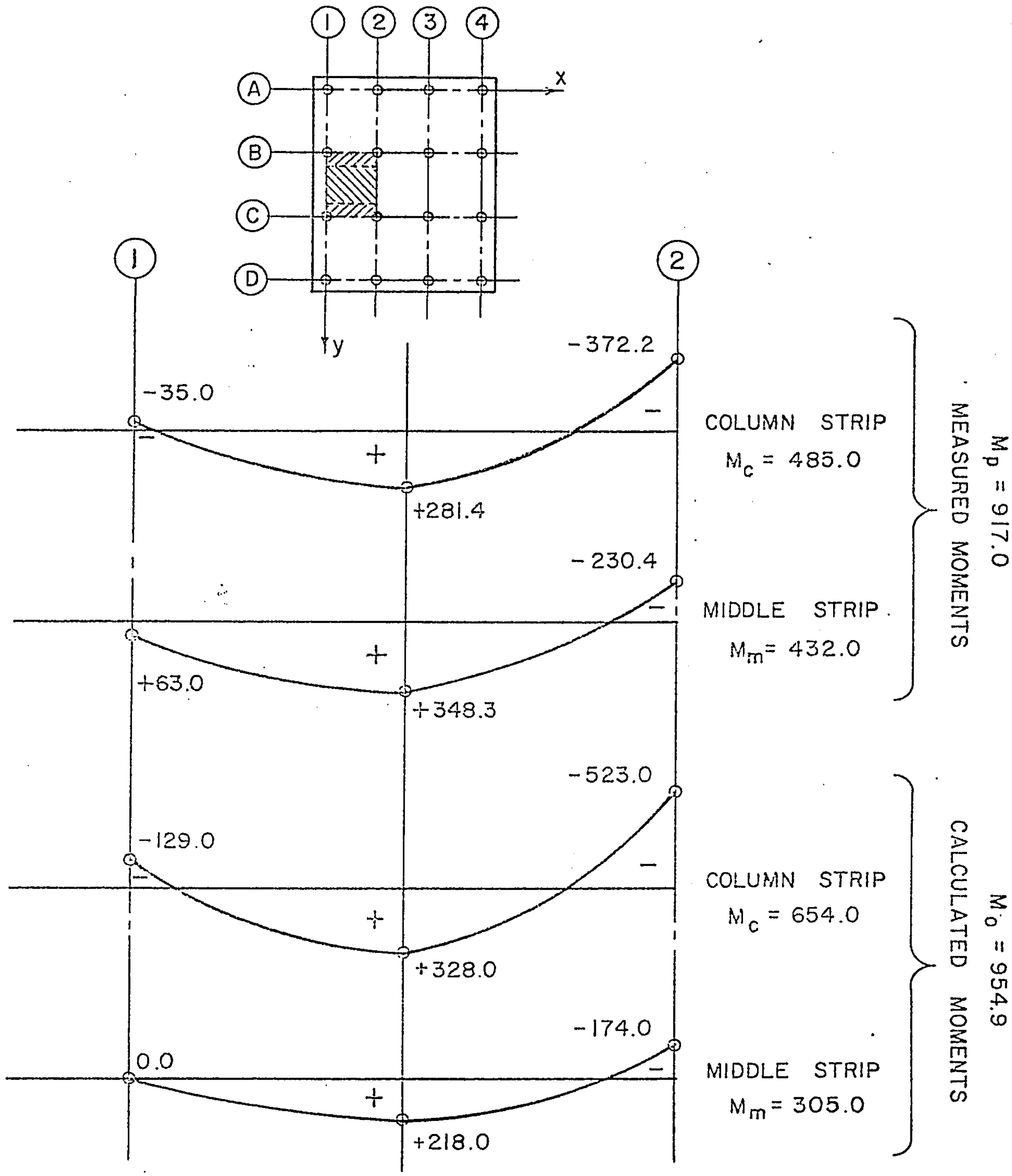
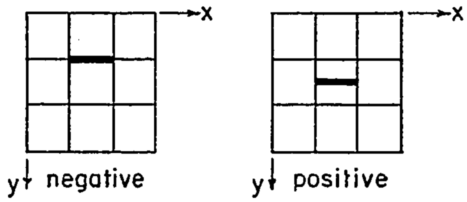


FIG. 5.22: COMPARISON OF CALCULATED MOMENTS AND MEASURED MOMENTS, FOR AN END PANEL IN X-DIRECTION. SECOND LEVEL.



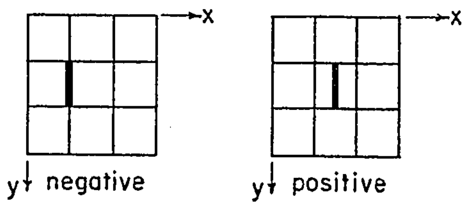
MOMENT SECTIONS
FIRST LEVEL

			-ve MOMENT		+ve MOMENT		M_o (lb.in.)	LINE No.
			COLUMN STRIP	MIDDLE STRIP	COLUMN STRIP	MIDDLE STRIP		
ACI 318-71 (ACI 318-63) MOMENTS	TOTAL -ve (or+ve) MOMENT	M (lb.in.)	-793		+389			1
			(-608.6)*		(+340.1)*			2
		% OF M_o [a]	71.5%		35%			3
		% OF (M_o)*	(68%)*		(38%)*			4
		(% OF M_o) [b]	(54.8%)		(30.6%)			5
	COL.(OR MID.) STRIP MOMENT	M (lb.in.)	-595	-198	+233	+157	1109.9	6
			(-447.5)*	(-161.1)*	(+196.9)*	(+143.2)*	(894.9)	7
		% OF -ve(or+ve) MOMENT	75%	25%	60%	40%		8
			(73.5%)*	(26.5%)*	(57.9%)*	(42.1%)*		9
		% OF M_o [a]	53.6%	17.9%	21%	14%		10
		% OF(M_o)*	(50%)*	(18%)*	(22%)*	(16%)*		11
		(% OF M_o) [b]	(40.3%)	(14.5%)	(17.7%)	(12.9%)		12
EXPERIMENTALLY MEASURED MOMENTS	TOTAT -ve(or +ve) MOMENT	M (lb.in.)	-667		+387.5			13
		% OF M_o	60.1%		34.9%			14
	COL.(OR MID.) STRIP MOMENT	M (lb.in.)	-496.	-171.	+201.3	+186.2		15
		% OF -ve(or+ve) MOMENT	74.4%	25.6%	52%	48%		16
		% OF M_o	44.7%	15.4%	18.1%	16.8%		17

TABLE. 5.1: COMPARISON OF ACI MOMENTS AND EXPERIMENTALLY MEASURED MOMENTS, (FOR AN INTERIOR PANEL IN Y-DIRECTION .FIRST LEVEL)

---

* ACI 318-63
a ACI 318-71
b (COLUMN AND MIDDLE STRIP MOMENTS BY ACI 318-63)/(M_o BY ACI 318-71)

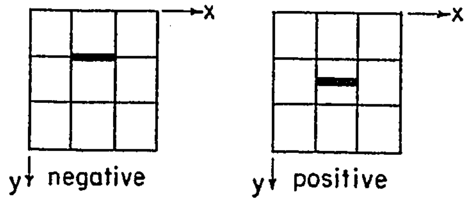


MOMENT SECTIONS
FIRST LEVEL

			-ve MOMENT COLUMN STRIP	-ve MOMENT MIDDLE STRIP	+ve MOMENT COLUMN STRIP	+ve MOMENT MIDDLE STRIP	M_o (lb.in.)	LINE No.
ACI 318-71 (ACI 318-63) MOMENTS	TOTAL -ve(or+ve) MOMENT	M (lb.in.)	-689 (-521.6)*		+334 (+291.5)*			1, 2
		% OF M_o [a]	72.2%		35%			3
		% OF (M_o)*	(68.0%)*		(38.0%)*			4
		(% OF M_o) [b]	(54.6%)		(30.5%)		954.9	5
	COL.(OR MID.) STRIP MOMENT	M (lb.in.)	-517 (-335.6)*	-172 (-186.0)*	+200 (+147.7)*	+134 (+143.8)*	(767.2)*	6, 7
		% OF -ve(or+ve) MOMENT	75% (64.3%)*	25% (35.7%)*	60% (50.7%)*	40% (49.3%)*		8, 9
		% OF M_o [a]	54.2%	18%	21%	14%		10
		% OF (M_o)*	(43.8%)*	(24.2%)*	(19.2%)*	(18.8%)*		11
		(% OF M_o) [b]	(35.1%)	(19.5%)	(15.5%)	(15.1%)		12
EXPERIMENTALLY MEASURED MOMENTS	TOTAT -ve(or+ve) MOMENT	M (lb.in.)	-600.6		+380.8			13
		% OF M_o	62.9%		39.9%			14
	COL.(OR MID.) STRIP MOMENT	M (lb.in.)	-409.1	-191.5	+176.4	+204.4		15
		% OF -ve(or+ve) MOMENT	68.1%	31.9%	46.3%	53.7%		16
		% OF M_o	42.8%	20.1%	18.5%	21.4%		17

TABLE.5.2: COMPARISON OF ACI MOMENTS AND EXPERIMENTALLY MEASURED MOMENTS, (FOR AN INTERIOR PANEL IN X-DIRECTION .FIRST LEVEL)

* ACI 318-63
a ACI 318-71
b (COLUMN AND MIDDLE STRIP MOMENTS BY ACI 318-63)/(M_o BY ACI 318-71)

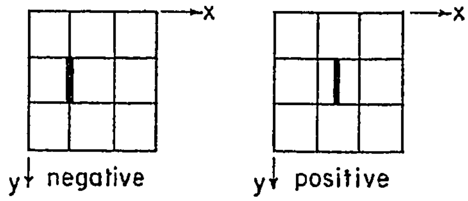


MOMENT SECTIONS
SECOND LEVEL

			−ve MOMENT COLUMN STRIP	−ve MOMENT MIDDLE STRIP	+ve MOMENT COLUMN STRIP	+ve MOMENT MIDDLE STRIP	M_o (lb.in.)	LINE No.
ACI 318-71 (ACI 318-63) MOMENTS	TOTAL −ve (or +ve) MOMENT	M (lb.in.)	−802.5		+389			1
			(−608.6)*		(+340.1)*			2
		% OF M_o [a]	72.3%		35%			3
		% OF (M_o)*	(68.%)*		(38.%)*			4
		(% OF M_o) [b]	(54.8%)		(30.6%)			5
	COL.(OR MID.) STRIP MOMENT	M (lb.in.)	−602	−200.5	+233	+156	1109.9	6
			(−447.5)*	(−161.1)*	(196.9)*	(+143.2)*	(894.9)	7
		% OF −ve (or +ve) MOMENT	75%	25%	60%	40%		8
			(73.5%)*	(26.5%)*	(57.9%)*	(42.1%)*		9
		% OF M_o [a]	54.2%	18.1%	21%	14%		10
		% OF (M_o)*	(50.%)*	(18.%)*	(22.%)*	(16.%)*		11
		(% OF M_o) [b]	(40.3%)	(14.5%)	(17.7%)	(12.9%)		12
EXPERIMENTALLY MEASURED MOMENTS	TOTAT −ve (or +ve) MOMENT	M (lb.in.)	−764.9		+308.7			13
		% OF M_o	68.9%		27.8%			14
	COL.(OR MID.) STRIP MOMENT	M (lb.in.)	−539.7	−225.2	+162.6	+146.1		15
		% OF −ve (or +ve) MOMENT	70.6%	29.4%	52.7%	47.3%		16
		% OF M_o	48.6%	20.3%	14.6%	13.2%		17

TABLE. 5.3: COMPARISON OF ACI MOMENTS AND EXPERIMENTALLY MEASURED MOMENTS, (FOR AN INTERIOR PANEL IN Y-DIRECTION . SECOND LEVEL)

* ACI 318-63
a ACI 318-71
b (COLUMN AND MIDDLE STRIP MOMENTS BY ACI 318-63)/(M_o BY ACI 318-71)



MOMENT SECTIONS SECOND LEVEL

			− ve MOMENT COLUMN STRIP	− ve MOMENT MIDDLE STRIP	+ ve MOMENT COLUMN STRIP	+ ve MOMENT MIDDLE STRIP	M_o (lb.in.)	LINE No.
ACI 318-71 (ACI 318-63) MOMENTS	TOTAL -ve (or +ve) MOMENT	M (lb.in.)	−697		+334			1
			(−521.6)*		(+291.5)*			2
		% OF M_o [a]	73%		35%			3
		% OF (M_o)*	(68.0%)*		(38.0%)*			4
		(% OF M_o) [b]	(54.6%)		(30.5%)			5
	COL. (OR MID.) STRIP MOMENT	M (lb.in.)	−523	−174	+200	+134	954.9	6
			(−335.6)*	(−186.0)*	(+147.7)*	(+143.8)*	(767.2)*	7
		% OF -ve (or +ve) MOMENT	75%	25%	60%	40%		8
			(64.3%)*	(35.7%)*	(50.7%)*	(49.3%)*		9
		% OF M_o [a]	54.8%	18.2%	21.%	14.%		10
		% OF (M_o)*	(43.8%)*	(24.3%)*	(19.3%)*	(18.8%)*		11
		(% OF M_o) [b]	(35.1%)	(19.5%)	(15.5%)	(15.1%)		12
EXPERIMENTALLY MEASURED MOMENTS	TOTAT -ve (or +ve) MOMENT	M (lb.in.)	−602.6		+284.5			13
		% OF M_o	63.1%		29.8%			14
	COL. (OR MID.) STRIP MOMENT	M (lb.in.)	−372.2	−230.4	+135.1	+149.4		15
		% OF -ve (or +ve) MOMENT	61.8%	38.2%	47.5%	52.5%		16
		% OF M_o	39.%	24.1%	14.1%	15.6%		17

TABLE. 5.4: COMPARISON OF ACI MOMENTS AND EXPERIMENTALLY MEASURED MOMENTS, (FOR AN INTERIOR PANEL IN X-DIRECTION. SECOND LEVEL.)

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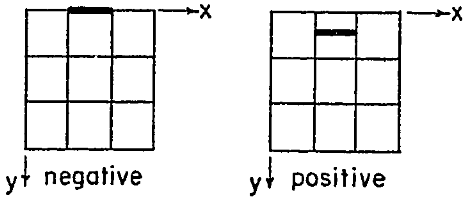
* ACI 318-63

a ACI 318-71

b (COLUMN AND MIDDLE STRIP MOMENTS BY ACI 318-63)/(M_o BY ACI 318-71)

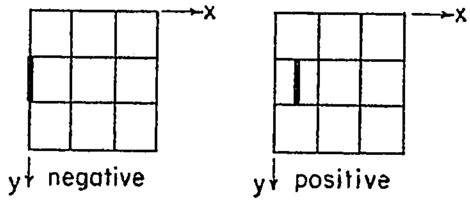


MOMENT SECTIONS
FIRST LEVEL

			-ve MOMENT COLUMN STRIP	-ve MOMENT MIDDLE STRIP	+ve MOMENT COLUMN STRIP	+ve MOMENT MIDDLE STRIP	M_o (lb.in.)	LINE No.
ACI 318-71 (ACI 318-63) MOMENTS	TOTAL -ve (or +ve) MOMENT	M (lb.in.)	-253		+590			1
			(-447.5)*		(+429.6)*			2
		% OF M_o a	22.8%		53.2%			3
		% OF (M_o)*	(50%)*		(48%)*			4
		(% OF M_o) b	(40.3%)		(38.7%)			5
	COL. (OR MID.) STRIP MOMENT	M (lb.in.)	-253	00.0	+354	+236	1109.9	6
			(-358.0)*	(-89.5)*	(+250.6)*	(-179.0)*	(894.9)*	7
		% OF -ve(or +ve) MOMENT	100%	0.0%	60%	40%		8
			(80%)*	(20%)*	(58.3%)*	(41.7%)*		9
		% OF M_o a	22.8%	0.0%	31.9%	21.3%		10
		% OF (M_o)*	(40%)	(10%)	(28%)	(20%)		11
		(% OF M_o) b	(32.2%)	(8.1%)	(22.6%)	(16.1%)		12
EXPERIMENTALLY MEASURED MOMENTS	TOTAT -ve(or +ve) MOMENT	M (lb.in.)	-56.3		+653.8			13
		% OF M_o	5.1%		58.9%			14
	COL. (OR MID.) STRIP MOMENT	M (lb.in.)	-104.3	+48.0	+324.8	+329		15
		% OF -ve(or +ve) MOMENT	–	–	49.7%	50.3%		16
		% OF M_o	9.4%	-4.3%	29.3%	29.6%		17

TABLE.5.5 : COMPARISON OF ACI MOMENTS AND EXPERIMENTALLY MEASURED MOMENTS, (FOR AN END PANEL IN Y-DIRECTION .FIRST LEVEL)

* ACI 318-63
a ACI 318-71
b (COLUMN AND MIDDLE STRIP MOMENTS BY ACI 318-63)/(M_o BY ACI 318-71)

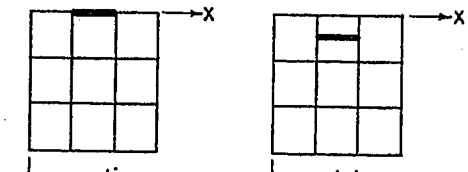


MOMENT SECTIONS
FIRST LEVEL

			-ve MOMENT		+ve MOMENT		M_o (lb.in.)	LINE No.
			COLUMN STRIP	MIDDLE STRIP	COLUMN STRIP	MIDDLE STRIP		
ACI 318-71 (ACI 318-63) MOMENTS	TOTAL -ve (or +ve) MOMENT	M (lb.in.)	-171		+528			1
			(-383.7)*		(+368.3)*			2
		% OF M_o [a]	17.9%		55.3%			3
		% OF (M_o)*	(50. %)*		(48. %)*			4
		(% OF M_o) [b]	(40.2%)		(38.6%)			5
	COL. (OR MID.) STRIP MOMENT	M (lb.in.)	-171	00.0	+317	+211	954.9	6
			(-268.6)*	(-115.1)*	(+188.0)*	(+180.3)*	(767.2)*	7
		% OF -ve (or +ve) MOMENT	100%	00.0%	60%	40%		8
			(70%)	(30%)	(51%)	(49%)		9
		% OF M_o [a]	17.9%	00.0%	33.2%	22.1%		10
		% OF (M_o)*	(35%)*	(15%)*	(24.5%)*	(23.5%)*		11
		(% OF M_o) [b]	(28.1%)	(12.1%)	(19.7%)	(18.9%)		12
EXPERIMENTALLY MEASURED MOMENTS	TOTAT -ve (or +ve) MOMENT	M (lb.in.)	-30.4		+617.5			13
		% OF M_o	3.2%		64.7%			14
	COL. (OR MID.) STRIP MOMENT	M (lb.in.)	-84.4	+54.	+280.	+337.5		15
		% OF -ve (or +ve) MOMENT	–	–	45.3%	54.7%		16
		% OF M_o	8.8%	- 5.7%	29.3%	35.3%		17

TABLE.5.6: COMPARISON OF ACI MOMENTS AND EXPERIMENTALLY MEASURED MOMENTS, (FOR AN END PANEL IN X-DIRECTION .FIRST LEVEL)

* ACI 318-63
a ACI 318-71
b (COLUMN AND MIDDLE STRIP MOMENTS BY ACI 318-63)/(M_o BY ACI 318-71)



negative MOMENT SECOND LEVEL · positive MOMENT SECTIONS LEVEL

			- ve MOMENT COLUMN STRIP	- ve MOMENT MIDDLE STRIP	+ve MOMENT COLUMN STRIP	+ve MOMENT MIDDLE STRIP	M_o (lb.in.)	LINE No.
ACI 318-71 (ACI 318-63) MOMENTS	TOTAL -ve (or+ve) MOMENT	M (lb.in.)	-192		+616			1
			(-447.5)*		(+429.6)*			2
		% OF M_o [a]	17.3%		55.5%			3
		% OF (M_o)*	(50%)*		(48%)*			4
		(% OF M_o) [b]	(40.3%)		(38.7%)			5
	COL. (OR MID.) STRIP MOMENT	M (lb.in.)	-192	00.0	+370	+246	1109.9	6
			(-358.)*	(-89.5)*	(+250.6)*	(-179.0)*	(894.9)*	7
		% OF -ve(or+ve) MOMENT	100%	00%	60%	40%		8
			(80%)*	(20%)*	(58.3%)*	(41.7%)*		9
		% OF M_o [a]	17.3%	0.0%	33.3%	22.2%		10
		% OF(M_o)*	(40%)*	(10%)*	(28%)*	(20%)*		11
		(% OF M_o) [b]	(32.2%)	(8.1%)	(22.6%)	(16.1%)		12
EXPERIMENTALLY MEASURED MOMENTS	TOTAT -ve (or +ve) MOMENT	M (lb.in.)	+12.6		+673.0			13
		% OF M_o	- 1.1%		60.6%			14
	COL. (OR MID.) STRIP MOMENT	M (lb.in.)	-43.4	+56.0	+336.	+337		15
		% OF -ve(or+ve) MOMENT	-	-	49.9%	50.1%		16
		% OF M_o	3.9%	-5%	30.3%	30.4%		17

TABLE. 5.7: COMPARISON OF ACI MOMENTS AND EXPERIMENTALLY MEASURED MOMENTS, (FOR AN END PANEL IN Y-DIRECTION. SECOND LEVEL.)

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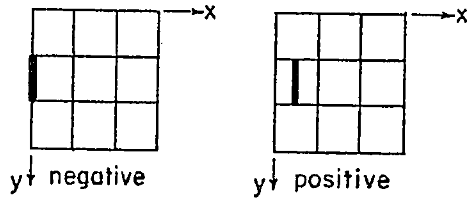
* ACI 318-63

a ACI 318-71

b (COLUMN AND MIDDLE STRIP MOMENTS BY ACI 318-63)/(M_o BY ACI 318-71)



MOMENT SECTIONS
SECOND LEVEL

			-ve MOMENT COLUMN STRIP	-ve MOMENT MIDDLE STRIP	+ve MOMENT COLUMN STRIP	+ve MOMENT MIDDLE STRIP	M_o (lb.in.)	LINE No.
ACI 318-71 (ACI 318-63) MOMENTS	TOTAL -ve (or +ve) MOMENT	M (lb.in.)	-129		+546			1
			(-383.7)*		(+368.3)*			2
		% OF M_o [a]	13.5%		57.2%			3
		% OF (M_o)*	(50%)*		(48.%)*			4
		(% OF M_o) [b]	(40.2%)		(38.6%)			5
	COL. (OR MID.) STRIP MOMENT	M (lb.in.)	-129	0.00	+328	+218	954.9	6
			(-268.6)*	(-115.1)*	(+188.0)*	(+180.3)*		7
		% OF -ve(or +ve) MOMENT	100%	00.0%	60%	40%		8
			(70%)*	(30%)*	(51%)*	(49%)*		9
		% OF M_o [a]	13.5%	00.%	34.3%	22.9%		10
		% OF (M_o)*	(35%)*	(15%)*	(24.5%)*	(23.5%)*		11
		(% OF M_o) [b]	(28.1%)	(12.1%)	(19.7%)	(18.9%)		12
EXPERIMENTALLY MEASURED MOMENTS	TOTAT -ve (or +ve) MOMENT	M (lb.in.)	+28.0		+629.7			13
		% OF M_o	- 2.9%		66.0%			14
	COL. (OR MID.) STRIP MOMENT	M (lb.in.)	-35.0	+63.0	+281.4	+348.3		15
		% OF -ve(or +ve) MOMENT	-	-	44.7%	55.3%		16
		% OF M_o	3.7%	- 6.6%	29.5%	36.5%		17

TABLE.5.8: COMPARISON OF ACI MOMENTS AND EXPERIMENTALLY MEASURED MOMENTS, (FOR AN END PANEL IN X-DIRECTION . SECOND LEVEL.)

* ACI 318-63
a ACI 318-71
b (COLUMN AND MIDDLE STRIP MOMENTS BY ACI 318-63)/(M_o BY ACI 318-71)

APPENDICES

APPENDIX A

EXPERIMENTAL RESULTS

STRAINS DUE TO UNIFORM LOAD = 200 PSF ON PANEL P1
LOAD NO 1 FIRST LEVEL

GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}	GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}
1	68.0	-139.0	32	33.0	-205.0
2	-20.0	-229.0	33	-78.0	-70.0
3	-100.0	-218.0	34	-259.0	-130.0
4	-100.0	-354.0	35	-266.0	-218.0
5	-118.0	476.0	36	-266.0	-322.0
6	-46.0	759.0	37	-313.0	444.0
7	-163.0	557.0	38	-81.0	636.0
8	-297.0	-353.0	39	-300.0	538.0
9	-297.0	128.0	40	-207.0	-270.0
10	76.0	-194.0	41	-207.0	160.0
11	81.0	-515.0	42	-104.0	-60.0
12	273.0	761.0	43	-126.0	-464.0
13	60.0	-331.0	44	-273.0	506.0
14	150.0	-197.0	45	-210.0	-100.0
15	576.0	-238.0	46	-74.0	-52.0
16	639.0	752.0	47	-63.0	-196.0
17	615.0	-104.0	48	-279.0	320.0
18	147.0	-198.0	49	-177.0	16.0
19	347.0	-334.0	50	-37.0	-52.0
20	647.0	-204.0	51	-50.0	-110.0
21	676.0	518.0	52	-44.0	-116.0
22	726.0	746.0	53	-135.0	106.0
23	686.0	573.0	54	-235.0	194.0
24	715.0	-50.0	55	-178.0	160.0
25	125.0	-178.0	56	-118.0	22.0
26	504.0	-244.0	57	-100.0	-286.0
27	600.0	766.0	58	-387.0	-270.0
28	564.0	-92.0	59	-147.0	-270.0
29	-2.0	-90.0	60	-297.0	-112.0
30	36.0	-546.0	61	-219.0	-55.0
31	135.0	700.0	62	-188.0	-55.0

TABLE A.1

STRAINS DUE TO UNIFORM LOAD = 200 PSF ON PANEL P2,
LOAD NO 2 FIRST LEVEL

GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}	GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}
1	14.0	-37.0	32	-213.0	-90.0
2	63.0	-36.0	33	-62.0	-70.0
3	9.0	43.0	34	-168.0	-140.0
4	9.0	-9.0	35	-153.0	-228.0
5	442.0	39.0	36	-153.0	-290.0
6	-65.0	64.0	37	-238.0	415.0
7	-7.0	65.0	38	-19.0	615.0
8	-3.0	12.0	39	-300.0	475.0
9	-3.0	60.0	40	-197.0	-260.0
10	20.0	-48.0	41	-197.0	110.0
11	6.0	-3.0	42	19.0	-100.0
12	0.0	87.0	43	39.0	-590.0
13	6.0	65.0	44	171.0	680.0
14	19.0	-44.0	45	-9.0	-290.0
15	-21.0	-42.0	46	101.0	-150.0
16	-69.0	150.0	47	489.0	-280.0
17	-81.0	59.0	48	576.0	710.0
18	-41.0	-49.0	49	546.0	-80.0
19	4.0	-84.0	50	110.0	-160.0
20	-52.0	-80.0	51	340.0	-300.0
21	-88.0	104.0	52	571.0	-240.0
22	-216.0	214.0	53	460.0	460.0
23	-165.0	170.0	54	677.0	690.0
24	-136.0	50.0	55	737.0	540.0
25	-8.0	-40.0	56	663.0	-60.0
26	-72.0	-160.0	57	9.0	17.0
27	-180.0	300.0	58	-229.0	-259.0
28	-189.0	-30.0	59	-75.0	-259.0
29	-45.0	62.0	60	-3.0	36.0
30	-102.0	-372.0	61	-266.0	-75.0
31	-192.0	480.0	62	-122.0	-75.0

TABLE A.2

STRAINS DUE TO UNIFORM LOAD = 200 PSF ON PANEL P3

LOAD NO 3 FIRST LEVEL

GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}	GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}
1	0.0	6.0	32	51.0	25.0
2	-7.0	11.0	33	24.0	-30.0
3	-14.0	20.0	34	47.0	-30.0
4	-14.0	-2.0	35	21.0	35.0
5	-69.0	9.0	36	21.0	-10.0
6	-5.0	12.0	37	35.0	20.0
7	-6.0	13.0	38	-28.0	42.0
8	-16.0	11.0	39	47.0	38.0
9	-16.0	7.0	40	41.0	20.0
10	0.0	4.0	41	41.0	40.0
11	-11.0	0.0	42	9.0	-34.0
12	-4.0	12.0	43	10.0	0.0
13	-16.0	-4.0	44	-7.0	72.0
14	2.0	-1.0	45	26.0	68.0
15	2.0	4.0	46	-20.0	-50.0
16	4.0	10.0	47	-33.0	-58.0
17	9.0	4.0	48	-88.0	135.0
18	13.0	-3.0	49	-52.0	70.0
19	11.0	0.0	50	-35.0	-50.0
20	5.0	5.0	51	-52.0	-94.0
21	14.0	5.0	52	-44.0	-105.0
22	2.0	5.0	53	-127.0	90.0
23	18.0	10.0	54	-194.0	210.0
24	19.0	5.0	55	-165.0	185.0
25	19.0	-5.0	56	-105.0	68.0
26	15.0	0.0	57	-14.0	9.0
27	13.0	5.0	58	32.0	12.0
28	37.0	20.0	59	8.0	12.0
29	28.0	15.0	60	-17.0	7.0
30	26.0	15.0	61	43.0	30.0
31	12.0	25.0	62	36.0	30.0

TABLE A.3

STRAINS DUE TO UNIFORM LOAD = 200 PSF ON PANEL P4
LOAD NO 4 FIRST LEVEL

GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}	GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}
1	-18.0	768.0	32	6.0	60.0
2	-136.0	488.0	33	-159.0	636.0
3	-109.0	-494.0	34	-348.0	460.0
4	-109.0	-194.0	35	-249.0	-360.0
5	-45.0	-213.0	36	-249.0	-60.0
6	69.0	-147.0	37	-210.0	-54.0
7	55.0	-61.0	38	-59.0	-20.0
8	21.0	63.0	39	-26.0	14.0
9	21.0	-13.0	40	2.0	68.0
10	258.0	922.0	41	2.0	50.0
11	96.0	-516.0	42	-270.0	500.0
12	100.0	-162.0	43	-144.0	-336.0
13	29.0	7.0	44	-78.0	-8.0
14	606.0	755.0	45	2.0	80.0
15	501.0	-237.0	46	-237.0	310.0
16	156.0	-190.0	47	-87.0	-110.0
17	48.0	-15.0	48	-65.0	10.0
18	680.0	732.0	49	-27.0	64.0
19	636.0	530.0	50	-207.0	220.0
20	577.0	-140.0	51	-121.0	136.0
21	325.0	-250.0	52	-57.0	-42.0
22	128.0	-140.0	53	-36.0	-30.0
23	117.0	-48.0	54	-37.0	14.0
24	19.0	44.0	55	-41.0	40.0
25	561.0	744.0	56	-18.0	66.0
26	462.0	-186.0	57	-107.0	-344.0
27	110.0	-120.0	58	-383.0	-210.0
28	32.0	700.0	59	-106.0	-210.0
29	78.0	30.0	60	20.0	25.0
30	63.0	-414.0	61	1.0	59.0
31	8.0	-50.0	62	1.0	59.0

TABLE A.4

STRAINS DUE TO UNIFORM LOAD = 200 PSF ON PANEL P5

LOAD NO 5 FIRST LEVEL

GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}	GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}
1	-30.0	24.0	32	-2.0	16.0
2	6.0	23.0	33	-62.0	564.0
3	25.0	-2.0	34	-288.0	380.0
4	25.0	27.0	35	-183.0	-442.0
5	15.0	-28.0	36	-183.0	-134.0
6	15.0	-34.0	37	-195.0	-100.0
7	6.0	-23.0	38	-53.0	-64.0
8	1.0	-2.0	39	-15.0	-30.0
9	1.0	0.0	40	5.0	22.0
10	-30.0	57.0	41	5.0	0.0
11	0.0	19.0	42	114.0	626.0
12	-7.0	-35.0	43	-18.0	-490.0
13	-10.0	7.0	44	-1.0	-92.0
14	-210.0	108.0	45	9.0	10.0
15	-36.0	-34.0	46	237.0	640.0
16	-17.0	-41.0	47	423.0	-230.0
17	-26.0	7.0	48	98.0	-144.0
18	-189.0	172.0	49	13.0	5.0
19	-115.0	84.0	50	634.0	640.0
20	-60.0	-74.0	51	578.0	440.0
21	-48.0	-74.0	52	528.0	-186.0
22	-43.0	-40.0	53	161.0	-274.0
23	-35.0	-22.0	54	98.0	-160.0
24	-28.0	10.0	55	83.0	-74.0
25	-213.0	260.0	56	-3.0	5.0
26	-87.0	-152.0	57	25.0	13.0
27	-63.0	-52.0	58	-244.0	-288.0
28	-29.0	446.0	59	-121.0	-288.0
29	-237.0	10.0	60	1.0	0.0
30	-117.0	-340.0	61	10.0	11.0
31	-80.0	-55.0	62	0.0	11.0

TABLE A.5

STRAINS DUE TO UNIFORM LOAD = 200 PSF ON PANEL P6

LOAD NO 6 FIRST LEVEL

GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}	GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}
1	-6.0	9.0	32	13.0	0.0
2	-2.0	7.0	33	-26.0	30.0
3	-15.0	-2.0	34	39.0	15.0
4	-15.0	8.0	35	28.0	-5.0
5	-34.0	5.0	36	28.0	22.0
6	0.0	0.0	37	45.0	-30.0
7	0.0	1.0	38	25.0	-30.0
8	-10.0	-5.0	39	25.0	-12.0
9	-10.0	3.0	40	8.0	6.0
10	-3.0	8.0	41	8.0	-3.0
11	-6.0	-1.0	42	-36.0	53.0
12	0.0	0.0	43	14.0	18.0
13	-7.0	-6.0	44	12.0	-38.0
14	-1.0	9.0	45	2.0	6.0
15	5.0	5.0	46	-120.0	108.0
16	12.0	-3.0	47	-30.0	-39.0
17	2.0	-4.0	48	-16.0	-40.0
18	0.0	8.0	49	-15.0	11.0
19	14.0	10.0	50	-199.0	182.0
20	5.0	10.0	51	-125.0	105.0
21	12.0	0.0	52	-57.0	-74.0
22	12.0	-5.0	53	-29.0	-78.0
23	10.0	-8.0	54	-35.0	47.0
24	4.0	-10.0	55	-36.0	-10.0
25	12.0	15.0	56	-23.0	15.0
26	13.0	16.0	57	-15.0	2.0
27	21.0	-14.0	58	35.0	9.0
28	9.0	22.0	59	20.0	9.0
29	16.0	0.0	60	-10.0	-1.0
30	25.0	30.0	61	12.0	2.0
31	29.0	-22.0	62	3.0	2.0

TABLE A.6

STRAINS DUE TO UNIFORM LOAD = 200 PSF ON PANEL P7

LOAD NO 7 FIRST LEVEL

GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}	GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}
1	69.0	-136.0	32	1.0	-17.0
2	63.0	-32.0	33	-62.0	-78.0
3	17.0	98.0	34	-35.0	-27.0
4	17.0	64.0	35	3.0	46.0
5	20.0	52.0	36	3.0	35.0
6	0.0	25.0	37	5.0	26.0
7	0.0	0.0	38	4.0	10.0
8	3.0	-39.0	39	3.0	0.0
9	3.0	10.0	40	0.0	-23.0
10	104.0	-179.0	41	0.0	14.0
11	23.0	79.0	42	-93.0	-62.0
12	8.0	22.0	43	2.0	70.0
13	9.0	-10.0	44	1.0	12.0
14	148.0	-182.0	45	11.0	-6.0
15	32.0	38.0	46	-70.0	-54.0
16	8.0	19.0	47	-31.0	42.0
17	10.0	-5.0	48	-2.0	12.0
18	124.0	-183.0	49	5.0	-8.0
19	104.0	-60.0	50	-34.0	-54.0
20	5.0	25.0	51	-41.0	-9.0
21	11.0	35.0	52	-22.0	30.0
22	4.0	20.0	53	2.0	26.0
23	10.0	0.0	54	-3.0	10.0
24	4.0	-8.0	55	0.0	-3.0
25	102.0	-165.0	56	4.0	-10.0
26	17.0	32.0	57	16.0	81.0
27	5.0	18.0	58	-3.0	41.0
28	5.0	-106.0	59	9.0	41.0
29	-7.0	-90.0	60	3.0	-14.0
30	4.0	52.0	61	-4.0	-5.0
31	4.0	11.0	62	3.0	-5.0

TABLE A.7

STRAINS DUE TO UNIFORM LOAD = 200 PSF ON PANEL P8

LOAD NO 8 FIRST LEVEL

GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}	GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}
1	12.0	-30.0	32	0.0	-7.0
2	9.0	-11.0	33	-52.0	-74.0
3	7.0	13.0	34	-19.0	-22.0
4	7.0	18.0	35	11.0	58.0
5	13.0	15.0	36	11.0	34.0
6	-3.0	7.0	37	19.0	26.0
7	-4.0	0.0	38	5.0	12.0
8	0.0	-19.0	39	3.0	8.0
9	0.0	2.0	40	-1.0	-16.0
10	2.0	-41.0	41	-1.0	6.0
11	-1.0	28.0	42	7.0	-104.0
12	-5.0	10.0	43	2.0	60.0
13	0.0	-3.0	44	6.0	13.0
14	-16.0	-39.0	45	0.0	-4.0
15	-14.0	28.0	46	100.0	-154.0
16	-4.0	12.0	47	10.0	34.0
17	0.0	-2.0	48	7.0	16.0
18	-32.0	-43.0	49	0.0	-2.0
19	-36.0	-4.0	50	77.0	-157.0
20	-24.0	30.0	51	82.0	-56.0
21	-15.0	26.0	52	-13.0	27.0
22	-2.0	12.0	53	9.0	34.0
23	-2.0	0.0	54	0.0	16.0
24	0.0	0.0	55	8.0	2.0
25	-60.0	-44.0	56	1.0	-2.0
26	-31.0	42.0	57	6.0	16.0
27	-1.0	12.0	58	18.0	46.0
28	0.0	-52.0	59	4.0	46.0
29	-86.0	-8.0	60	0.0	-8.0
30	-3.0	66.0	61	-2.0	-5.0
31	2.0	16.0	62	1.0	-5.0

TABLE A.8

STRAINS DUE TO UNIFORM LOAD = 200 PSF ON PANEL P9

LOAD NO 9 FIRST LEVEL

GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}	GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}
1	-4.0	0.0	32	0.0	12.0
2	7.0	1.0	33	21.0	-20.0
3	-13.0	-13.0	34	35.0	0.0
4	-13.0	3.0	35	9.0	30.0
5	-8.0	0.0	36	9.0	23.0
6	-1.0	0.0	37	25.0	23.0
7	-1.0	0.0	38	2.0	16.0
8	-5.0	-11.0	39	-2.0	13.0
9	-5.0	5.0	40	0.0	0.0
10	4.0	0.0	41	0.0	15.0
11	4.0	-5.0	42	19.0	-28.0
12	0.0	5.0	43	13.0	38.0
13	0.0	-4.0	44	0.0	14.0
14	9.0	0.0	45	0.0	3.0
15	10.0	0.0	46	-11.0	-34.0
16	0.0	3.0	47	-7.0	40.0
17	0.0	0.0	48	0.0	25.0
18	7.0	-3.0	49	0.0	10.0
19	20.0	0.0	50	-33.0	-40.0
20	2.0	26.0	51	-29.0	0.0
21	13.0	18.0	52	-22.0	40.0
22	0.0	8.0	53	0.0	32.0
23	3.0	8.0	54	-4.0	22.0
24	0.0	6.0	55	0.0	13.0
25	20.0	0.0	56	1.0	9.0
26	17.0	24.0	57	-13.0	-5.0
27	0.0	14.0	58	15.0	27.0
28	0.0	-19.0	59	2.0	27.0
29	28.0	0.0	60	-5.0	-3.0
30	20.0	34.0	61	1.0	7.0
31	0.0	14.0	62	-1.0	7.0

TABLE A.9

STRAINS DUE TO UNIFORM LOAD = 200 PSF ON PANEL P1
LOAD NO 10 SECOND LEVEL

GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}	GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}
1	96.0	-204.0	32	87.0	-170.0
2	27.0	-329.0	33	-75.0	-90.0
3	34.0	-382.0	34	-293.0	-170.0
4	34.0	-356.0	35	-260.0	-290.0
5	-140.0	601.0	36	-260.0	-355.0
6	-3.0	983.0	37	-345.0	500.0
7	-52.0	811.0	38	-178.0	745.0
8	13.0	-8.0	39	-355.0	640.0
9	13.0	3.0	40	-315.0	110.0
10	126.0	-227.0	41	-315.0	50.0
11	330.0	-606.0	42	-117.0	-70.0
12	360.0	962.0	43	-138.0	-520.0
13	270.0	-173.0	44	-345.0	620.0
14	186.0	-253.0	45	-300.0	-50.0
15	675.0	-283.0	46	-78.0	-65.0
16	723.0	932.0	47	-81.0	-225.0
17	720.0	0.0	48	-315.0	405.0
18	179.0	-250.0	49	-216.0	70.0
19	433.0	-385.0	50	-41.0	-70.0
20	691.0	-240.0	51	-60.0	-130.0
21	780.0	575.0	52	-62.0	-140.0
22	775.0	905.0	53	-133.0	140.0
23	856.0	750.0	54	-247.0	290.0
24	787.0	10.0	55	-222.0	240.0
25	132.0	-220.0	56	-145.0	85.0
26	558.0	-282.0	57	34.0	-369.0
27	600.0	885.0	58	-344.0	-323.0
28	630.0	-146.0	59	-179.0	-323.0
29	-3.0	0.0	60	13.0	-4.0
30	42.0	-575.0	61	-320.0	80.0
31	180.0	805.0	62	-310.0	80.0

TABLE A.10

STRAINS DUE TO UNIFORM LOAD = 200 PSF ON PANEL P2

LOAD NO 11 SECOND LEVEL

GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}	GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}
1	16.0	-55.0	32	-243.0	-130.0
2	4.0	-54.0	33	-53.0	-103.0
3	9.0	9.0	34	-220.0	-170.0
4	9.0	-4.0	35	-180.0	-270.0
5	404.0	53.0	36	-180.0	-380.0
6	-48.0	102.0	37	-255.0	425.0
7	-22.0	114.0	38	-55.0	656.0
8	-8.0	82.0	39	-275.0	572.0
9	-8.0	60.0	40	-260.0	75.0
10	-2.0	-57.0	41	-260.0	28.0
11	-15.0	-17.0	42	15.0	-135.0
12	-69.0	122.0	43	60.0	-620.0
13	-9.0	104.0	44	90.0	742.0
14	-18.0	-62.0	45	12.0	-115.0
15	-48.0	-63.0	46	116.0	-205.0
16	-129.0	196.0	47	450.0	-310.0
17	-90.0	96.0	48	525.0	765.0
18	-26.0	-69.0	49	528.0	-20.0
19	-59.0	-132.0	50	91.0	-212.0
20	-46.0	-122.0	51	316.0	-348.0
21	-158.0	125.0	52	530.0	-255.0
22	-201.0	270.0	53	412.0	486.0
23	-200.0	248.0	54	661.0	775.0
24	-132.0	88.0	55	718.0	640.0
25	-57.0	-87.0	56	627.0	-5.0
26	-96.0	-215.0	57	9.0	2.0
27	-252.0	380.0	58	-181.0	-325.0
28	-204.0	-97.0	59	-180.0	-325.0
29	-90.0	62.0	60	-8.0	71.0
30	-144.0	-459.0	61	-241.0	52.0
31	-237.0	545.0	62	-276.0	52.0

TABLE A.11

STRAINS DUE TO UNIFORM LOAD = 200 PSF ON PANEL P3

LOAD NO 12 SECOND LEVEL

GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}	GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}
1	0.0	0.0	32	46.0	55.0
2	4.0	4.0	33	17.0	-45.0
3	-2.0	5.0	34	45.0	-54.0
4	-2.0	4.0	35	21.0	-7.0
5	-73.0	9.0	36	21.0	-9.0
6	-6.0	13.0	37	35.0	28.0
7	1.0	12.0	38	-39.0	51.0
8	-1.0	2.0	39	50.0	62.0
9	-1.0	0.0	40	54.0	40.0
10	5.0	-1.0	41	54.0	12.0
11	-1.0	0.0	42	17.0	-41.0
12	0.0	15.0	43	9.0	-9.0
13	3.0	-2.0	44	0.0	87.0
14	10.0	-7.0	45	29.0	80.0
15	8.0	0.0	46	15.0	-60.0
16	6.0	13.0	47	-36.0	-75.0
17	17.0	4.0	48	-90.0	166.0
18	9.0	-13.0	49	-71.0	87.0
19	23.0	0.0	50	-41.0	-60.0
20	3.0	3.0	51	-52.0	-115.0
21	28.0	5.0	52	-50.0	-118.0
22	1.0	11.0	53	-45.0	118.0
23	30.0	20.0	54	-230.0	245.0
24	26.0	8.0	55	-192.0	235.0
25	20.0	-10.0	56	-141.0	92.0
26	20.0	0.0	57	-2.0	4.0
27	27.0	22.0	58	21.0	0.0
28	38.0	-30.0	59	20.0	0.0
29	29.0	18.0	60	-1.0	1.0
30	31.0	12.0	61	46.0	26.0
31	24.0	33.0	62	61.0	26.0

TABLE A.12

STRAINS DUE TO UNIFORM LOAD = 200 PSF ON PANEL P4

LOAD NO 13 SECOND LEVEL

GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}	GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}
1	11.0	795.0	32	7.0	-15.0
2	-47.0	516.0	33	-157.0	600.0
3	0.0	-404.0	34	-376.0	405.0
4	0.0	-296.0	35	-286.0	-225.0
5	-98.0	-267.0	36	-286.0	-180.0
6	93.0	-192.0	37	-270.0	-130.0
7	89.0	-100.0	38	-66.0	-87.0
8	45.0	4.0	39	-43.0	-55.0
9	45.0	-12.0	40	6.0	-10.0
10	312.0	802.0	41	6.0	0.0
11	249.0	-479.0	42	-315.0	480.0
12	138.0	-207.0	43	-183.0	-395.0
13	61.0	-9.0	44	-96.0	-70.0
14	669.0	784.0	45	-21.0	7.0
15	594.0	-215.0	46	-249.0	282.0
16	192.0	-226.0	47	-108.0	-165.0
17	67.0	-31.0	48	-78.0	-62.0
18	716.0	765.0	49	-40.0	3.0
19	688.0	515.0	50	-227.0	182.0
20	618.0	-180.0	51	-165.0	87.0
21	395.0	-335.0	52	-70.0	-95.0
22	164.0	-225.0	53	-34.0	-96.0
23	156.0	-125.0	54	-44.0	-59.0
24	41.0	-45.0	55	-55.0	-28.0
25	561.0	700.0	56	-23.0	0.0
26	474.0	-205.0	57	0.0	-350.0
27	126.0	-195.0	58	-363.0	-203.0
28	34.0	675.0	59	-209.0	-203.0
29	141.0	-40.0	60	45.0	-4.0
30	33.0	-430.0	61	1.0	-5.0
31	-12.0	-130.0	62	11.0	-5.0

TABLE A.13

STRAINS DUE TO UNIFORM LOAD = 200 PSF ON PANEL P5
LOAD NO 14 SECOND LEVEL

GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}	GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}
1	-32.0	46.0	32	-13.0	22.0
2	0.0	32.0	33	-63.0	563.0
3	3.0	0.0	34	-272.0	388.0
4	3.0	2.0	35	-206.0	-276.0
5	277.0	-46.0	36	-206.0	-210.0
6	13.0	-49.0	37	-190.0	-124.0
7	3.0	-33.0	38	-53.0	-86.0
8	1.0	-5.0	39	-24.0	-44.0
9	1.0	-2.0	40	11.0	0.0
10	-39.0	64.0	41	11.0	0.0
11	-18.0	3.0	42	150.0	628.0
12	-3.0	-51.0	43	36.0	-452.0
13	-13.0	3.0	44	3.0	-109.0
14	-96.0	121.0	45	7.0	7.0
15	-45.0	-41.0	46	516.0	676.0
16	-24.0	-53.0	47	435.0	-222.0
17	-27.0	3.0	48	96.0	-172.0
18	-185.0	185.0	49	11.0	-4.0
19	-137.0	98.0	50	632.0	668.0
20	-58.0	-80.0	51	600.0	465.0
21	-62.0	-94.0	52	522.0	-182.0
22	-38.0	-62.0	53	165.0	-290.0
23	-48.0	-25.0	54	93.0	-179.0
24	-29.0	2.0	55	93.0	-90.0
25	-198.0	267.0	56	-2.0	-10.0
26	-99.0	-140.0	57	3.0	1.0
27	-66.0	-66.0	58	-205.0	-243.0
28	-28.0	446.0	59	-208.0	-243.0
29	-237.0	10.0	60	1.0	-3.0
30	-165.0	-348.0	61	10.0	0.0
31	-90.0	-72.0	62	13.0	0.0

TABLE A.14

STRAINS DUE TO UNIFORM LOAD = 200 PSF ON PANEL P6

LOAD NO 15 SECOND LEVEL

GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}	GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}
1	-5.0	10.0	32	10.0	5.0
2	-5.0	9.0	33	-34.0	33.0
3	-7.0	1.0	34	19.0	19.0
4	-7.0	4.0	35	22.0	6.0
5	-48.0	2.0	36	22.0	22.0
6	2.0	1.0	37	37.0	-40.0
7	-2.0	-2.0	38	21.0	-40.0
8	0.0	-4.0	39	23.0	-23.0
9	0.0	-2.0	40	12.0	0.0
10	-5.0	10.0	41	12.0	0.0
11	0.0	2.0	42	-50.0	59.0
12	0.0	-2.0	43	0.0	10.0
13	0.0	-3.0	44	1.0	-50.0
14	-4.0	10.0	45	2.0	5.0
15	1.0	6.0	46	-140.0	122.0
16	1.0	-6.0	47	-57.0	-45.0
17	5.0	-3.0	48	-22.0	-50.0
18	-2.0	9.0	49	-28.0	5.0
19	4.0	5.0	50	-219.0	190.0
20	3.0	8.0	51	-152.0	116.0
21	12.0	0.0	52	-72.0	-84.0
22	11.0	0.0	53	-50.0	-92.0
23	14.0	-4.0	54	-44.0	-49.0
24	6.0	0.0	55	-54.0	-22.0
25	8.0	10.0	56	-30.0	7.0
26	10.0	8.0	57	-7.0	2.0
27	9.0	-13.0	58	28.0	14.0
28	10.0	18.0	59	15.0	14.0
29	11.0	0.0	60	0.0	-3.0
30	19.0	25.0	61	12.0	0.0
31	16.0	-34.0	62	12.0	0.0

TABLE A.15

STRAINS DUE TO UNIFORM LOAD = 200 PSF ON PANEL P7
LOAD NO 16 SECOND LEVEL

GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}	GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}
1	90.0	-182.0	32	3.0	9.0
2	77.0	-47.0	33	-59.0	-84.0
3	18.0	111.0	34	-48.0	-28.0
4	18.0	73.0	35	8.0	32.0
5	-2.0	65.0	36	8.0	41.0
6	-2.0	42.0	37	4.0	41.0
7	2.0	21.0	38	2.0	30.0
8	-1.0	0.0	39	0.0	20.0
9	-1.0	3.0	40	1.0	7.0
10	93.0	-196.0	41	1.0	5.0
11	32.0	92.0	42	-102.0	-62.0
12	8.0	36.0	43	-10.0	68.0
13	5.0	-1.0	44	0.0	28.0
14	180.0	-219.0	45	3.0	0.0
15	40.0	37.0	46	-78.0	-55.0
16	6.0	32.0	47	-32.0	40.0
17	7.0	2.0	48	-2.0	24.0
18	150.0	-219.0	49	2.0	7.0
19	127.0	-73.0	50	-31.0	-55.0
20	8.0	35.0	51	-49.0	0.0
21	23.0	50.0	52	-25.0	34.0
22	2.0	40.0	53	0.0	40.0
23	13.0	24.0	54	-3.0	25.0
24	2.0	7.0	55	-5.0	19.0
25	144.0	-80.0	56	5.0	8.0
26	18.0	30.0	57	18.0	92.0
27	2.0	34.0	58	5.0	37.0
28	5.0	-125.0	59	12.0	37.0
29	-9.0	12.0	60	-1.0	1.0
30	2.0	55.0	61	1.0	6.0
31	2.0	26.0	62	2.0	6.0

TABLE A.16

STRAINS DUE TO UNIFORM LOAD = 200 PSF ON PANEL P8

LOAD NO 17 SECOND LEVEL

GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}	GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}
1	15.0	-60.0	32	1.0	0.0
2	5.0	-23.0	33	-52.0	-95.0
3	3.0	21.0	34	-24.0	-50.0
4	3.0	11.0	35	20.0	25.0
5	13.0	16.0	36	20.0	23.0
6	-4.0	10.0	37	13.0	16.0
7	-3.0	5.0	38	4.0	6.0
8	-10.0	-3.0	39	4.0	3.0
9	-10.0	-3.0	40	-3.0	0.0
10	2.0	-57.0	41	-3.0	0.0
11	-6.0	25.0	42	0.0	-130.0
12	-1.0	13.0	43	2.0	44.0
13	-1.0	-5.0	44	4.0	0.0
14	-19.0	-60.0	45	2.0	-16.0
15	-26.0	25.0	46	80.0	-186.0
16	-1.0	16.0	47	16.0	10.0
17	0.0	-1.0	48	4.0	6.0
18	-40.0	-64.0	49	4.0	0.0
19	-46.0	-20.0	50	97.0	-196.0
20	-28.0	20.0	51	92.0	-84.0
21	-18.0	12.0	52	-8.0	7.0
22	-5.0	5.0	53	6.0	24.0
23	-4.0	0.0	54	0.0	7.0
24	-1.0	0.0	55	9.0	0.0
25	-62.0	-80.0	56	4.0	0.0
26	-31.0	30.0	57	3.0	16.0
27	0.0	15.0	58	22.0	24.0
28	1.0	-80.0	59	17.0	24.0
29	-83.0	-4.0	60	-10.0	-3.0
30	-15.0	62.0	61	-3.0	0.0
31	2.0	10.0	62	-3.0	0.0

TABLE A.17

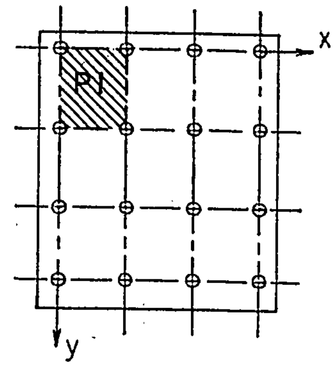
STRAINS DUE TO UNIFORM LOAD = 200 PSF ON PANEL P9

LOAD NO 18 SECOND LEVEL

GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}	GAGE NO	STRAINS X-DIR. IN X 10^{-6}	STRAINS Y-DIR. IN X 10^{-6}
1	0.0	0.0	32	-2.0	2.0
2	8.0	1.0	33	21.0	-37.0
3	0.0	4.0	34	35.0	-5.0
4	0.0	1.0	35	12.0	19.0
5	-16.0	5.0	36	12.0	11.0
6	0.0	4.0	37	21.0	16.0
7	3.0	4.0	38	2.0	15.0
8	1.0	3.0	39	-2.0	9.0
9	1.0	1.0	40	0.0	8.0
10	6.0	-1.0	41	0.0	2.0
11	3.0	2.0	42	13.0	-48.0
12	2.0	6.0	43	8.0	37.0
13	-4.0	3.0	44	6.0	16.0
14	12.0	-6.0	45	-2.0	9.0
15	6.0	4.0	46	-15.0	-53.0
16	4.0	6.0	47	-13.0	40.0
17	-3.0	3.0	48	0.0	28.0
18	14.0	-10.0	49	-1.0	8.0
19	22.0	6.0	50	-44.0	-52.0
20	3.0	15.0	51	-47.0	-3.0
21	10.0	13.0	52	-27.0	44.0
22	0.0	11.0	53	-2.0	39.0
23	5.0	14.0	54	-3.0	31.0
24	1.0	7.0	55	-4.0	18.0
25	28.0	-8.0	56	2.0	13.0
26	12.0	19.0	57	0.0	2.0
27	6.0	12.0	58	11.0	15.0
28	-3.0	-28.0	59	14.0	15.0
29	30.0	8.0	60	1.0	2.0
30	12.0	30.0	61	-6.0	5.0
31	8.0	10.0	62	7.0	5.0

TABLE A.18

LOAD No: I
FIRST LEVEL

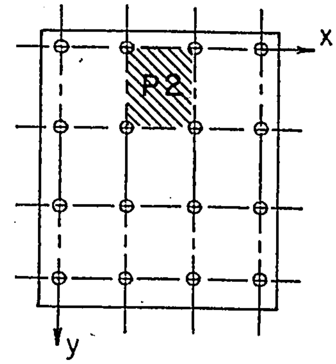


GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}	
	ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y
63		+ 68	74		+ 90	85	-690		96	- 50	
64		-478	75		-490	86	+280		97	- 99	
65		-676	76		+ 60	87	- 42		98	-170	
66		+346	77		+554	88	-410		99	+ 22	
67		-652	78		-1052	89	-610		100	-490	
68		+220	79		-300	90	+240		101	+530	
69		-582	80		+324	91	-543				
70		-1060	81		+620	92	- 68				
71		-140	82		-600	93	+210		18T		+188
									22T		-720
72		-446	83	- 85	[illegible]	94	-680		50T		+ 38
73		-574	84	-370	[illegible]	95	-370		54T		-670

TABLE A.19 STRAINS DUE TO UNIFORMLY DISTRIBUTED. LOAD = 200 P.S.F. ON MARKED PANELS.

NOTE : FOR LOCATIONS OF STRAIN GAGES SEE FIGS. 4.13 TO 4.16

LOAD No: 2
FIRST LEVEL

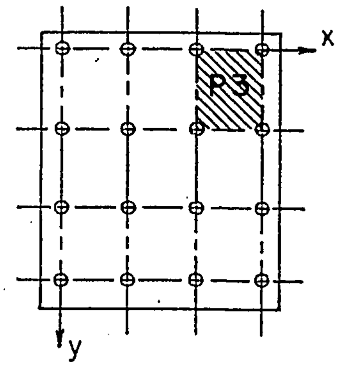


GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}	
	ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y
63		+ 70	74		+ 30	85	+ 33		96	-290	
64		- 65	75		-450	86	- 10		97	-260	
65		- 48	76		- 40	87	+100		98	- 20	
66		+ 80	77		+410	88	- 60		99	- 16	
67		- 80	78		-880	89	+ 80		100	+ 16	
68		+100	79		- 60	90	- 60		101	- 14	
69		+ 75	80		+ 70	91	-208				
70		- 75	81		+ 90	92	-300				
71		-110	82		- 70	93	-190		18T		+ 49
									22T		-230
72		-450	83	+50		94	-240		50T		+160
73		-590	84	-33		95	-125		54T		-200

TABLE A.20 STRAINS DUE TO UNIFORMLY DISTRIBUTED. LOAD = 200 P.S.F. ON MARKED PANELS.

NOTE : FOR LOCATIONS OF STRAIN GAGES SEE FIGS. 4.13 TO 4.16

LOAD No: 3
FIRST LEVEL

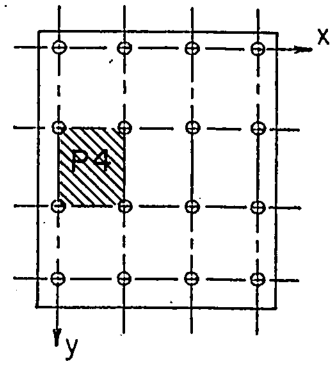


GAGE No.	STRAINS IN $X 10^{-6}$		GAGE No.	STRAINS IN $X 10^{-6}$		GAGE No.	STRAINS IN $X 10^{-6}$		GAGE No.	STRAINS IN $X 10^{-6}$	
	ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y
63		+60	74		+92	85	-17		96	000	
64		-40	75		+32	86	+23		97	+50	
65		-10	76		+20	87	+78		98	000	
66		+20	77		+88	88	-75		99	000	
67		-30	78		-25	89	-24		100	-15	
68		+12	79		-30	90	+37		101	+15	
69		+ 5	80		+30	91	+75				
70		-10	81		-15	92	-42				
71		+60	82		+15	93	000		18T		+10
									22T		000
72		-50	83	+49		94	+50		50T		+44
73		-28	84	-58		95	+50		54T		-258

TABLE A.21 STRAINS DUE TO UNIFORMLY DISTRIBUTED. LOAD = 200 P.S.F. ON MARKED PANELS.

NOTE : FOR LOCATIONS OF STRAIN GAGES SEE FIGS. 4.13 TO 4.16

LOAD No: 4
FIRST LEVEL

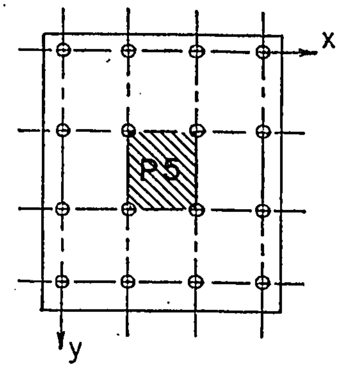


GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}	
	ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y
63		−480	74		−850	85	−660		96	+15	
64		+126	75		+ 46	86	+327		97	+19	
65		+422	76		+76	87	+ 91		98	000	
66		−882	77		000	88	− 56		99	000	
67		+ 52	78		+116	89	− 58		100	− 6	
68		+ 86	79		+370	90	+100		101	+ 5	
69		−24	80		−274	91	−495				
70		+162	81		− 36	92	− 28				
71		−510	82		+140	93	+286		18T		−673
									22T		+246
72		+ 64	83	− 35		94	−815		50T	−	−152
73		+506	84	−366		95	000		54T		+ 78

TABLE A.22 STRAINS DUE TO UNIFORMLY DISTRIBUTED. LOAD = 200 P.S.F. ON MARKED PANELS.

NOTE : FOR LOCATIONS OF STRAIN GAGES SEE FIGS. 4.13 TO 4.16

LOAD No: 5
FIRST LEVEL

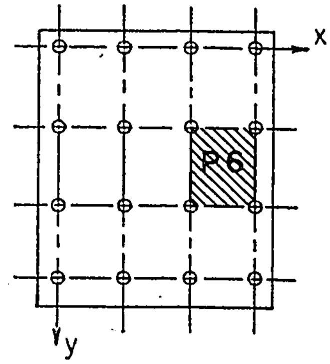


GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}	
	ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y
63		– 50	74		–850	85	+ 45		96	–10	
64		+ 86	75		– 10	86	– 20		97	000	
65		+ 50	76		+ 46	87	+ 42		98	+15	
66		– 16	77		– 34	88	– 36		99	–18	
67		– 30	78		+ 64	89	+ 8		100	+18	
68		+ 48	79		+ 80	90	– 8		101	– 5	
69		– 20	80		– 50	91	–225				
70		+ 20	81		000	92	–285				
71		–516	82		+ 22	93	–240		18T		–184
									22T		+ 56
72		– 22	83	+27		94	–345		50T		–578
73		+306	84	+15		95	+ 24		54T		+170

TABLE A.23 STRAINS DUE TO UNIFORMLY DISTRIBUTED. LOAD = 200 P.S.F. ON MARKED PANELS.

NOTE : FOR LOCATIONS OF STRAIN GAGES SEE FIGS. 4.13 TO 4.16

LOAD No: 6
FIRST LEVEL

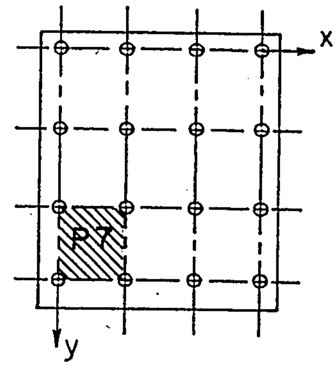


GAGE No.	STRAINS IN $\times 10^{-6}$		GAGE No.	STRAINS IN $\times 10^{-6}$		GAGE No.	STRAINS IN $\times 10^{-6}$		GAGE No.	STRAINS IN $\times 10^{-6}$	
	ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y
63		-24	74		+ 8	85	-22		96	-30	
64		+25	75		000	86	+11		97	+ 8	
65		+11	76		+20	87	+42		98	+ 5	
66		- 5	77		- 8	88	-58		99	-18	
67		-26	78		+48	89	-13		100	-19	
68		+33	79		+ 4	90	+10		101	+18	
69		+10	80		000	91	+60				
70		-11	81		000	92	-16				
71		-12	82		+ 8	93	+27		18T		- 3
									22T		+ 20
72		+70	83	+51		94	+19		50T		+178
73		+34	84	-55		95	+20		54T		+ 62

TABLE A.24 STRAINS DUE TO UNIFORMLY DISTRIBUTED. LOAD = 200 P.S.F. ON MARKED PANELS.

NOTE : FOR LOCATIONS OF STRAIN GAGES SEE FIGS. 4.13 TO 4.16

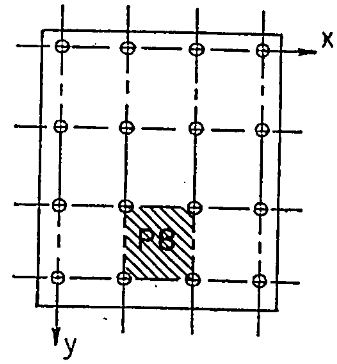
LOAD No: 7
FIRST LEVEL



GAGE No.	STRAINS IN X 10^{-6}		GAGE No.	STRAINS IN X 10^{-6}		GAGE No.	STRAINS IN X 10^{-6}		GAGE No.	STRAINS IN X 10^{-6}	
	ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y
63		- 10	74		+60	85	000		96	000	
64		+ 56	75		-60	86	+47		97	000	
65		- 52	76		+66	87	000		98	000	
66		+120	77		+24	88	000		99	+ 5	
67		- 50	78		-38	89	000		100	+15	
68		+ 50	79		-32	90	000		101	-10	
69		+ 26	80		+22	91	-15				
70		- 40	81		+75	92	+46				
71		- 28	82		-72	93	+ 5		18T		+193
									22T		- 24
72		+ 80	83	000		94	+26		50T		+ 44
73		- 25	84	+44		95	-25		54T		- 10

TABLE A.25 STRAINS DUE TO UNIFORMLY DISTRIBUTED. LOAD = 200 P.S.F. ON MARKED PANELS.

NOTE : FOR LOCATIONS OF STRAIN GAGES SEE FIGS. 4.13 TO 4.16



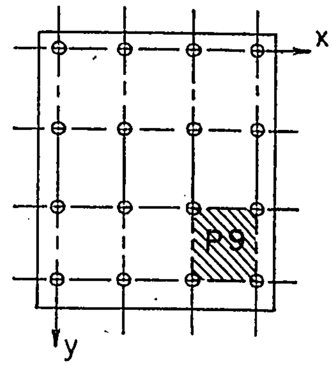
LOAD No: 8
FIRST LEVEL

GAGE No.	STRAINS IN $\times 10^{-6}$		GAGE No.	STRAINS IN $\times 10^{-6}$		GAGE No.	STRAINS IN $\times 10^{-6}$		GAGE No.	STRAINS IN $\times 10^{-6}$	
	ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y
63		-40	74		+72	85	+25		96	000	
64		+54	75		-50	86	-27		97	000	
65		+ 6	76		+41	87	- 3		98	000	
66		+20	77		+20	88	000		99	000	
67		-40	78		-26	89	+ 4		100	000	
68		+40	79		-18	90	000		101	+ 6	
69		+20	80		+ 8	91	+ 4				
70		-28	81		+34	92	+25				
71		+ 4	82		-24	93	000		18T		+ 48
									22T		- 6
72		+44	83	- 8		94	+46		50T		+170
73		-28	84	+30		95	- 6		54T		- 10

TABLE A.26 STRAINS DUE TO UNIFORMLY DISTRIBUTED. LOAD = 200 P.S.F. ON MARKED PANELS.

NOTE : FOR LOCATIONS OF STRAIN GAGES SEE FIGS. 4.13 TO 4.16

LOAD No: 9
FIRST LEVEL

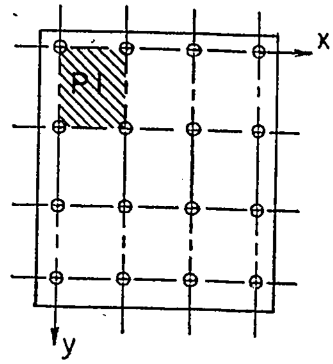


GAGE No.	STRAINS IN $\times 10^{-6}$		GAGE No.	STRAINS IN $\times 10^{-6}$		GAGE No.	STRAINS IN $\times 10^{-6}$		GAGE No.	STRAINS IN $\times 10^{-6}$	
	ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y
63		−16	74		+50	85	−15		96	− 8	
64		+44	75		−21	86	000		97	000	
65		+24	76		+41	87	+10		98	000	
66		+15	77		+29	88	−28		99	−21	
67		−20	78		− 7	89	−15		100	−19	
68		+40	79		+40	90	000		101	+10	
69		+21	80		− 7	91	+24				
70		+ 3	81		+35	92	−20				
71		−12	82		000	93	000		18T		+ 6
									22T		+20
72		+41	83	+40		94	+ 6		50T		+59
73		− 5	84	−39		95	+ 6		54T		000

TABLE A.27 STRAINS DUE TO UNIFORMLY DISTRIBUTED. LOAD = 200 P.S.F. ON MARKED PANELS.

NOTE : FOR LOCATIONS OF STRAIN GAGES SEE FIGS. 4.13 TO 4.16

LOAD No: 10
SECOND LEVEL

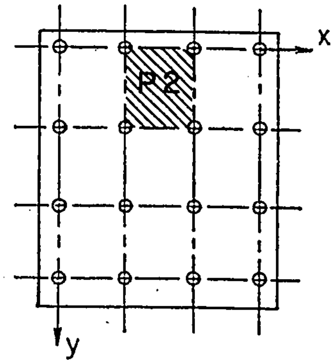


GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}	
	$\in_x$	$\in_y$		$\in_x$	$\in_y$		$\in_x$	$\in_y$		$\in_x$	$\in_y$
63		-285	74		-475	85	- 55		96	-265	
64		-190	75		-275	86	-360		97	-265	
65		-105	76		-200	87	-355		98	-167	
66		-360	77		-216	88	- 43		99	-150	
67		-175	78		-162	89	-166		100	- 50	
68		-240	79		-364	90	-175		101	-346	
69		-115	80		-165	91	-365				
70		-265	81		-243	92	-200				
71		-400	82		-150	93	- 80				
									22T		-830
72		-225	83	-315	[illegible]	94	-472		50T		+ 75
73		-100	84	-170	[illegible]	95	-230		54T		-322

TABLE A.28 STRAINS DUE TO UNIFORMLY DISTRIBUTED. LOAD = 200 P.S.F. ON MARKED PANELS.

NOTE : FOR LOCATIONS OF STRAIN GAGES SEE FIGS. 4.13 TO 4.16

LOAD No: 11
SECOND LEVEL

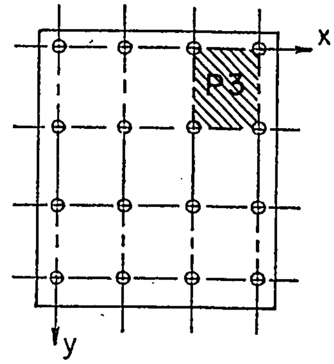


GAGE No.	STRAINS IN $\times 10^{-6}$		GAGE No.	STRAINS IN $\times 10^{-6}$		GAGE No.	STRAINS IN $\times 10^{-6}$		GAGE No.	STRAINS IN $\times 10^{-6}$	
	ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y
63		+ 20	74		-475	85	+ 13		96	-240	
64		+ 6	75		-215	86	000		97	-214	
65		+ 7	76		-200	87	-102		98	-170	
66		+ 15	77		-180	88	+120		99	- 50	
67.		+ 80	78		-153	89	+ 27		100	+ 80	
68		- 48	79		+ 30	90	- 45		101	+ 26	
69		- 3	80		- 4	91	-312				
70		+ 31	81		- 31	92	-196				
71		-356	82		+ 58	93	- 58				
									22T		-318
72		-232	83	000		94	-373		50T		+205
73		- 55	84	+ 20		95	-185		54T		-732

TABLE A.29 STRAINS DUE TO UNIFORMLY DISTRIBUTED. LOAD = 200 P.S.F. ON MARKED PANELS.

NOTE: FOR LOCATIONS OF STRAIN GAGES SEE FIGS. 4.13 TO 4.16

LOAD No: 12
SECOND LEVEL

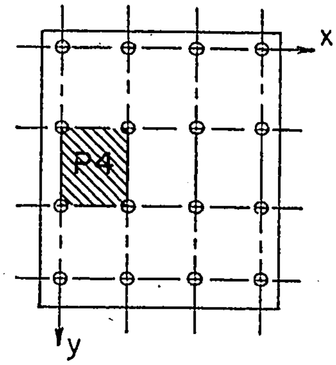


GAGE No.	STRAINS IN X 10^{-6}		GAGE No.	STRAINS IN X 10^{-6}		GAGE No.	STRAINS IN X 10^{-6}		GAGE No.	STRAINS IN X 10^{-6}	
	$\in_x$	$\in_y$		$\in_x$	$\in_y$		$\in_x$	$\in_y$		$\in_x$	$\in_y$
63		+ 4	74		+ 22	85	000		96	+ 41	
64		+ 5	75		+ 27	86	− 8		97	+ 29	
65		+ 3	76		+ 13	87	− 70		98	+ 16	
66		000	77		+ 10	88	+ 62		99	+ 6	
67		+ 12	78		+ 30	89	+ 10		100	000	
68		− 23	79		+ 6	90	− 16		101	000	
69		+ 5	80		000	91	− 22				
70		− 10	81		− 10	92	+ 30				
71		+ 10	82		+ 7	93	+ 6				
									22T		− 15
72		000	83	− 20		94	+ 3		50T		+ 55
73		000	84	+ 20		95	+ 3		54T		−290

TABLE A.30 STRAINS DUE TO UNIFORMLY DISTRIBUTED. LOAD = 200 P.S.F. ON MARKED PANELS.

NOTE : FOR LOCATIONS OF STRAIN GAGES SEE FIGS. 4.13 TO 4.16

LOAD No: 13
SECOND LEVEL

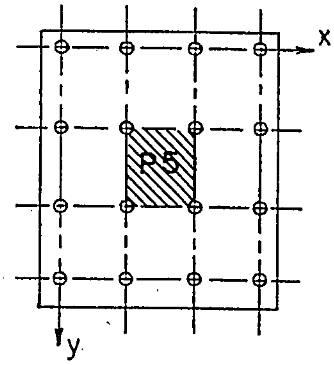


GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}	
	ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y
63		-290	74		-352	85	-130		96	+ 60	
64		-165	75		+ 12	86	-235		97	+ 40	
65		- 85	76		000	87	- 74		98	- 30	
66		-348	77		000	88	+ 93		99	+390	
67		+ 50	78		+ 9	89	+ 54		100	+ 80	
68		- 29	79		-345	90	- 50		101	- 35	
69		+ 8	80		-135	91	-325				
70		+ 21	81		+ 50	92	-206				
71		-312	82		000	93	-110				
									22T		+288
72		-200	83	-280		94	-374		50T		-212
73		-100	84	-190		95	- 40		54T		+ 50

TABLE A.31 STRAINS DUE TO UNIFORMLY DISTRIBUTED. LOAD = 200 P.S.F. ON MARKED PANELS.

NOTE : FOR LOCATIONS OF STRAIN GAGES SEE FIGS. 4.13 TO 4.16

LOAD No: 14
SECOND LEVEL

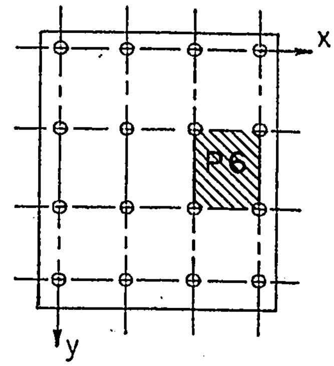


GAGE No.	STRAINS IN $\times 10^{-6}$		GAGE No.	STRAINS IN $\times 10^{-6}$		GAGE No.	STRAINS IN $\times 10^{-6}$		GAGE No.	STRAINS IN $\times 10^{-6}$	
	ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y
63		+ 45	74		-370	85	+ 15		96	+80	
64		- 12	75		+ 38	86	+ 6		97	+50	
65		- 8	76		+ 3	87	- 63		98	-40	
66		+ 42	77		+ 11	88	+ 62		99	000	
67		+ 41	78		+ 15	89	+ 20		100	+30	
68		- 40	79		+ 7	90	- 32		101	-30	
69		000	80		+ 31	91	-315				
70		+ 12	81		+ 3	92	-180				
71		-282	82		+ 9	93	-100				
									22T		+ 61
72		-210	83	-40		94	-340		50T		-600
73		- 80	84	+72		95	- 60		54T		+197

TABLE A.32 STRAINS DUE TO UNIFORMLY DISTRIBUTED. LOAD = 200 P.S.F. ON MARKED PANELS.

NOTE : FOR LOCATIONS OF STRAIN GAGES SEE FIGS. 4.13 TO 4.16

LOAD No: 15
SECOND LEVEL

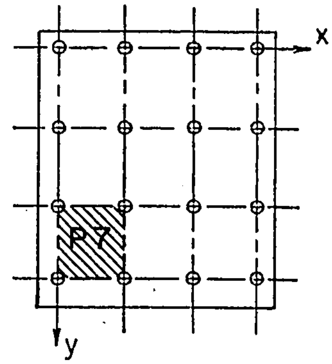


GAGE No.	STRAINS IN X10⁻⁶		GAGE No.	STRAINS IN X10⁻⁶		GAGE No.	STRAINS IN X10⁻⁶		GAGE No.	STRAINS IN X10⁻⁶	
	ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y
63		+18	74		+42	85	+ 3		96	+32	
64		-10	75		+10	86	- 3		97	+25	
65		000	76		+13	87	-30		98	-14	
66		000	77		+ 5	88	+22		99	+20	
67		+ 5	78		+ 3	89	+ 9		100	+ 4	
68		000	79		+ 2	90	-10		101	000	
69		+10	80		+12	91	-10				
70		000	81		000	92	+44				
71		+38	82		+20	93	-15				
									22T		+ 14
72		000	83	-24		94	+20		50T		-200
73		+ 5	84	+27		95	-23		54T		+ 46

TABLE A.33 STRAINS DUE TO UNIFORMLY DISTRIBUTED. LOAD = 200 P.S.F. ON MARKED PANELS.

NOTE : FOR LOCATIONS OF STRAIN GAGES SEE FIGS. 4.13 TO 4.16

LOAD No: 16
SECOND LEVEL

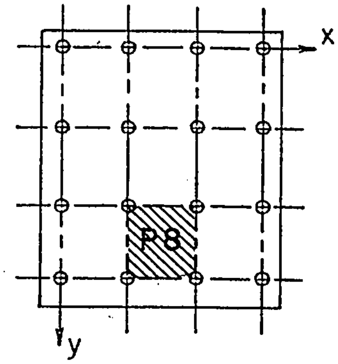


GAGE No.	STRAINS IN $\times 10^{-6}$ $\in_x$	STRAINS IN $\times 10^{-6}$ $\in_y$	GAGE No.	STRAINS IN $\times 10^{-6}$ $\in_x$	STRAINS IN $\times 10^{-6}$ $\in_y$	GAGE No.	STRAINS IN $\times 10^{-6}$ $\in_x$	STRAINS IN $\times 10^{-6}$ $\in_y$	GAGE No.	STRAINS IN $\times 10^{-6}$ $\in_x$	STRAINS IN $\times 10^{-6}$ $\in_y$
63		+32	74		+31	85	+29		96	-15	
64		+31	75		+21	86	+11		97	- 3	
65		+21	76		-18	87	+44		98	000	
66		+52	77		000	88	-38		99	+18	
67		+36	78		000	89	- 5		100	-28	
68		-40	79		+50	90	000		101	+21	
69		000	80		+50	91	+19				
70		+14	81		+10	92	+18				
71		+14	82		000	93	+19				
									22T		-28
72		+25	83	+33		94	+ 3		50T		+60
73		+ 9	84	+29		95	+ 1		54T		-37

TABLE A.34 STRAINS DUE TO UNIFORMLY DISTRIBUTED. LOAD = 200 P.S.F. ON MARKED PANELS.

NOTE: FOR LOCATIONS OF STRAIN GAGES SEE FIGS. 4.13 TO 4.16

LOAD No: 17
SECOND LEVEL

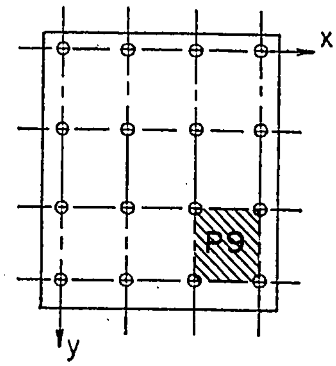


GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}		GAGE No.	STRAINS IN X10^{-6}	
	ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y
63		-15	74		+17	85	+ 8		96	- 5	
64		+ 6	75		000	86	000		97	000	
65		000	76		000	87	+26		98	000	
66		000	77		000	88	-26		99	-36	
67		000	78		- 6	89	000		100	-29	
68		-15	79		- 4	90	000		101	+21	
69		- 8	80		000	91	+20				
70		-12	81		000	92	+20				
71		000	82		-20	93	+ 5				
									22T		- 35
72		+27	83	+ 7		94	+26		50T		+182
73		000	84	000		95	+15		54T		- 24

TABLE A.35 STRAINS DUE TO UNIFORMLY DISTRIBUTED. LOAD = 200 P.S.F. ON MARKED PANELS.

NOTE : FOR LOCATIONS OF STRAIN GAGES SEE FIGS. 4.13 TO 4.16

LOAD No: 18
SECOND LEVEL



GAGE No.	STRAINS IN X 10^{-6}		GAGE No.	STRAINS IN X 10^{-6}		GAGE No.	STRAINS IN X 10^{-6}		GAGE No.	STRAINS IN X 10^{-6}	
	ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y		ϵ_x	ϵ_y
63		− 2	74		+16	85	−10		96	−30	
64		+14	75		−36	86	+ 5		97	000	
65		+13	76		+45	87	+30		98	000	
66		000	77		+18	88	−30		99	000	
67		−20	78		000	89	−11		100	− 6	
68		+41	79		+20	90	+ 8		101	000	
69		+12	80		000	91	+34				
70		000	81		+18	92	−11				
71		+ 2	82		+ 4	93	000				
									22T		+ 5
72		+21	83	+16		94	+22		50T		+70
73		+16	84	−24		95	+35		54T		− 8

TABLE A.36 STRAINS DUE TO UNIFORMLY DISTRIBUTED. LOAD = 200 P.S.F. ON MARKED PANELS.

NOTE : FOR LOCATIONS OF STRAIN GAGES SEE FIGS. 4.13 TO 4.16

GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}
1	+ 44	5	+58	9	-16
2	- 9	6	+12	10	- 2
3	+ 65	7	+48	11	- 8
4	+111	8	- 3	12	- 9

LOAD No: 1
FIRST LEVEL

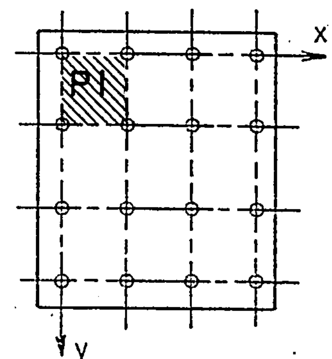


TABLE A.37: DEFLECTIONS DUE TO UNIFORMLY DISTRIBUTED LOAD OF 200 PSF ON PANEL No. "P1".

GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}
1	-10	5	+53	9	-3
2	+39	6	+92	10	-8
3	+ 4	7	- 3	11	-8
4	+12	8	+33	12	-3

LOAD No: 2
FIRST LEVEL

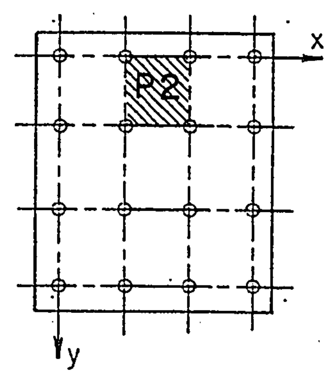


TABLE A.38: DEFLECTIONS DUE TO UNIFORMLY DISTRIBUTED LOAD OF 200 PSF ON PANEL No. "P2".

GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}
1	00	5	+ 1	9	00
2	- 9	6	+13	10	- 1
3	00	7	00	11	- 3
4	+ 2	8	- 4	12	- 8

LOAD No: 3
FIRST LEVEL

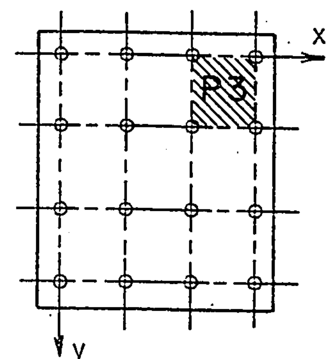


TABLE A.39: DEFLECTIONS DUE TO UNIFORMLY DISTRIBUTED LOAD OF 200 PSF ON PANEL No. "P3".

GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}
1	+ 1	5	- 7	9	+59
2	- 2	6	- 7	10	+97
3	-15	7	+37	11	+49
4	+ 1	8	- 4	12	+ 8

LOAD No: 4
FIRST LEVEL

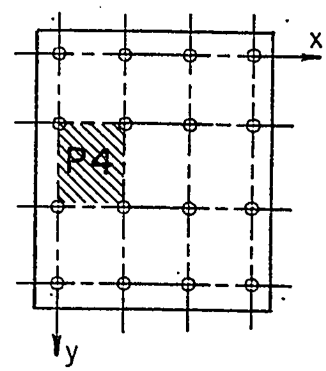


TABLE A.40: DEFLECTIONS DUE TO UNIFORMLY DISTRIBUTED LOAD OF 200 PSF ON PANEL No. "P4".

GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}
1	-2	5	- 8	9	+ 1
2	-1	6	- 1	10	+ 7
3	-3	7	- 4	11	+49
4	-7	8	+34	12	+82

LOAD No: 5
FIRST LEVEL

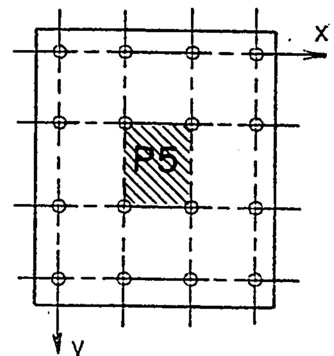


TABLE A.41: DEFLECTIONS DUE TO UNIFORMLY DISTRIBUTED LOAD OF 200 PSF ON PANEL No. "P5".

GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}
1	- 1	5	- 4	9	00
2	- 2	6	- 7	10	+ 2
3	00	7	00	11	- 2
4	00	8	- 5	12	+ 6

LOAD No: 6
FIRST LEVEL

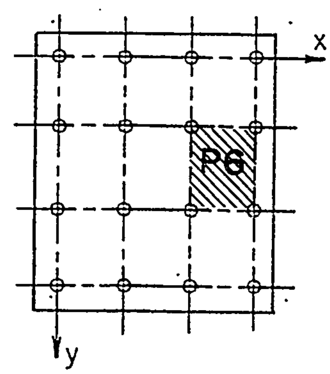


TABLE A.42: DEFLECTIONS DUE TO UNIFORMLY DISTRIBUTED LOAD OF 200 PSF ON PANEL No. "P6".

GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}
1	00	5	+ 2	9	-15
2	00	6	+ 2	10	- 5
3	+ 3	7	00	11	- 8
4	+ 3	8	- 2	12	- 9

LOAD No: 7
FIRST LEVEL

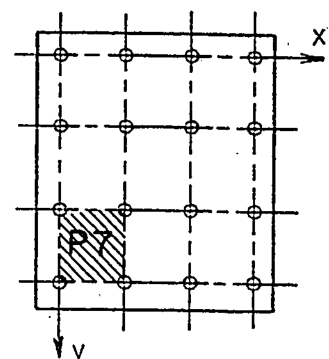


TABLE A.43: DEFLECTIONS DUE TO UNIFORMLY DISTRIBUTED LOAD OF 200 PSF ON PANEL No. "P7".

GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}
1	00	5	+2	9	-3
2	00	6	+2	10	-9
3	+ 1	7	-4	11	-8
4	+ 1	8	-1	12	-2

LOAD No: 8
FIRST LEVEL

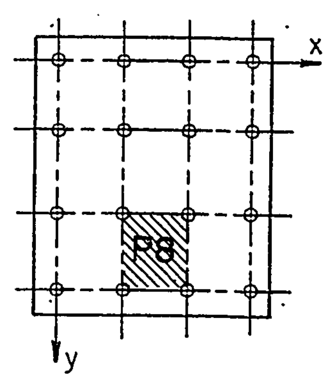


TABLE A.44: DEFLECTIONS DUE TO UNIFORMLY DISTRIBUTED LOAD OF 200 PSF ON PANEL No. "P8".

GAGE No.	DEFLECTION IN X10^{-3}	GAGE No.	DEFLECTION IN X10^{-3}	GAGE No.	DEFLECTION IN X10^{-3}
1	00	5	+1	9	00
2	00	6	+1	10	- 1
3	00	7	-1	11	- 4
4	+ 1	8	-2	12	- 9

LOAD No: 9
FIRST LEVEL

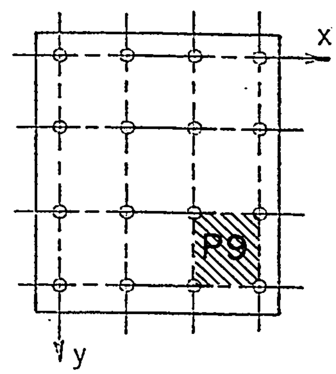


TABLE A.45: DEFLECTIONS DUE TO UNIFORMLY DISTRIBUTED LOAD OF 200 PSF ON PANEL No. "P9".

GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}
1	+ 52	5	+70	9	-25
2	= 12	6	+16	10	- 4
3	+ 90	7	+54	11	-12
4	+132	8	- 2	12	-12

LOAD No: 10
SECOND LEVEL

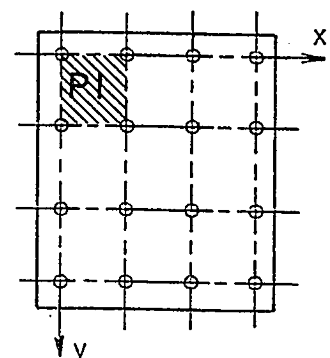


TABLE A.46: DEFLECTIONS DUE TO UNIFORMLY DISTRIBUTED LOAD OF 200 PSF ON PANEL No. "P1".

GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}
1	-11	5	+ 65	9	- 5
2	+50	6	+112	10	-11
3	+ 9	7	- 2	11	-13
4	+17	8	+ 42	12	- 5

LOAD No: 11
SECOND LEVEL

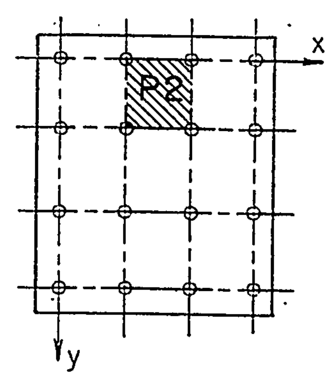


TABLE A.47: DEFLECTIONS DUE TO UNIFORMLY DISTRIBUTED LOAD OF 200 PSF ON PANEL No. "P2".

GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}
1	+ 2	5	+ 5	9	00
2	-14	6	+14	10	- 1
3	+ 1	7	+ 1	11	- 7
4	+ 3	8	- 6	12	-11

LOAD No: 12
SECOND LEVEL

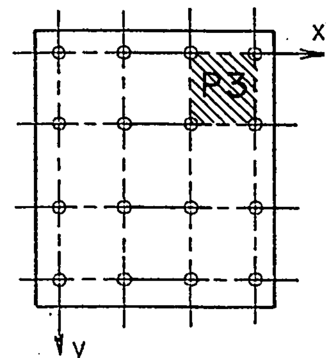


TABLE A.48: DEFLECTIONS DUE TO UNIFORMLY DISTRIBUTED LOAD OF 200 PSF ON PANEL No. "P3".

GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}
1	- 3	5	-11	9	+ 65
2	- 3	6	-12	10	+108
3	-21	7	+41	11	+ 55
4	- 4	8	- 5	12	+10

LOAD No: 13
SECOND LEVEL

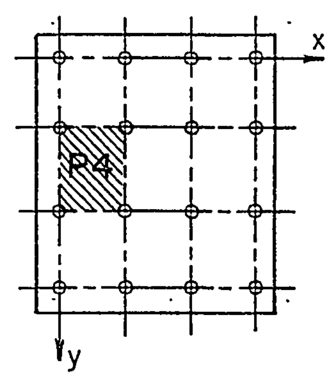


TABLE A.49: DEFLECTIONS DUE TO UNIFORMLY DISTRIBUTED LOAD OF 200 PSF ON PANEL No. "P4".

GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}
1	- 2	5	-12	9	+ 4
2	00	6	- 5	10	+10
3	- 5	7	- 5	11	+49
4	-11	8	+35	12	+91

LOAD No: 14
SECOND LEVEL

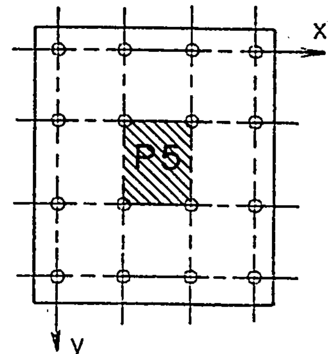


TABLE A.50: DEFLECTIONS DUE TO UNIFORMLY DISTRIBUTED LOAD OF 200 PSF ON PANEL No. "P5".

GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}
1	00	5	- 6	9	00
2	- 2	6	-11	10	+ 1
3	00	7	+ 1	11	+ 2
4	- 1	8	- 7	12	+ 8

LOAD No: 15
SECOND LEVEL

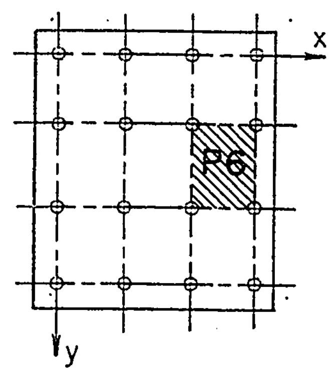


TABLE A.51: DEFLECTIONS DUE TO UNIFORMLY DISTRIBUTED LOAD OF 200 PSF ON PANEL No. "P6".

GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}
1	00	5	+3	9	-22
2	00	6	+2	10	- 4
3	+ 5	7	+1	11	-11
4	+ 4	8	-1	12	-12

LOAD No: 16
SECOND LEVEL

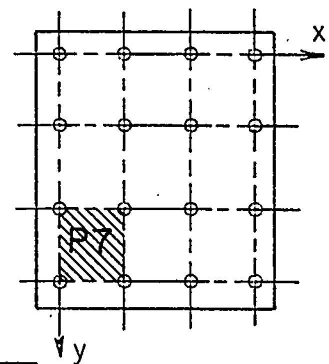


TABLE A.52: DEFLECTIONS DUE TO UNIFORMLY DISTRIBUTED LOAD OF 200 PSF ON PANEL No. "P7".

GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}	GAGE No.	DEFLECTION IN X 10^{-3}
1	00	5	+3	9	- 5
2	00	6	+2	10	-13
3	+ 2	7	-2	11	-13
4	+ 2	8	-1	12	- 6

LOAD No: 17
SECOND LEVEL

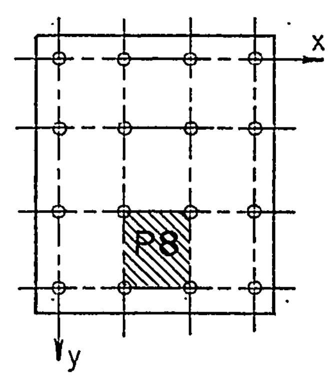


TABLE A.53: DEFLECTIONS DUE TO UNIFORMLY DISTRIBUTED LOAD OF 200 PSF ON PANEL No. "P8".

GAGE No.	DEFLECTION IN X10^{-3}	GAGE No.	DEFLECTION IN X10^{-3}	GAGE No.	DEFLECTION IN X10^{-3}
1	00	5	+ 2	9	- 1
2	00	6	+ 1	10	- 2
3	00	7	00	11	- 5
4	+ 1	8	- 3	12	-12

LOAD No: 18
SECOND LEVEL

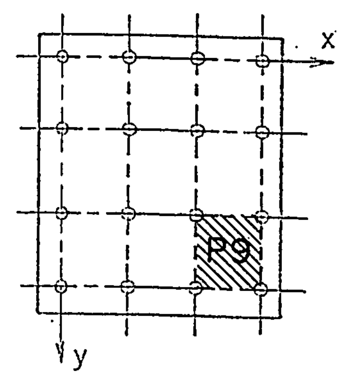


TABLE A.54: DEFLECTIONS DUE TO UNIFORMLY DISTRIBUTED LOAD OF 200 PSF ON PANEL No. "P9".

APPENDIX B

COMPUTER PROGRAM

```
$JOB   SK411310,FALTAOUS,PAGES=80,TIME=120,LINES=12000
C
C      ANALYSIS OF FLAT PLATES
C
C      EXPERIMENTAL INVESTIGATION
C
C
C      THIS PROGRAM USES EXPERIMENTALLY MEASURED STRAINS FOR
C      THE CALCULATIONS OF MOMENTS IN THE 'X' AND 'Y' DIRECTIONS
C      FOR DIFFERENT LOADING CASES
C
C
C                  MODEL M1
C                  ==========
C
C                  FIRST LEVEL
C                  =============
C
C
C      EXPLANATION OF SYMBOLS USED
C      ----------------------------
C
C      L(62,4,9)=LOAD NO1 TO LOAD NO 9
C                 FIRST COLUMN REPRESENTS THE LOAD NO.,
C                 SECOND COLUMN REPRESENTS STRAIN GAGE NUMBER,
C                 THIRD AND FORTH COLUMNS ARE THE STRAINS IN THE 'X' AND
C                 'Y' DIRECTIONS
C
C      A(62,3)   =STRAINS DUE TO PATTERN LOADING NO'1'
C                 (L.L.=200 PSF ON PANELS P1,P2,P3,P7,P8,P9)
C
C      B(62,3)   =STRAINS DUE TO PATTERN LOADING NO'2'
C                 (L.L.=200 PSF ON PANELS P2,P3,P4)
C
C      C(62,3)   =STRAINS DUE TO PATTERN LOADING NO'3'
C                 (L.L.=200 PSF ON PANEL P1,P3,P4,P6,P7,P9)
C
C      D(62,3)   =STRAINS DUE TO PATTERN LOADING NO'4'
C                 (L.L.=200 PSF ON PANELS P2,P5,P8)
C
C      E(62,3)   =STRAINS DUE TO PATTERN LOADING NO'5'
C                 (L.L.=200 PSF ON PANELS P1,P2,P3,P4,P5,P6)
C
C      F(62,3)   =STRAINS DUE TO PATTERN LOADING NO'6'
C                 (L.L.=200 PSF ON PANELS P1,P2,P4,P5,P7,P8)
C
C
C      T(62,3)   =STRAINS DUE TO UNIFORM LOAD=200 PSF ON ALL PANELS
C
C
```

```
C       M(62,5)=STRAINS AND MOMENTS  DE TO PATTERN LOADING NO 1    FOR:
C                L.L./D.L. =0.25,0.333,0.5,1.0,2.0,3.0,4.0,3.643
C
C       O(62,5)=STRAINS AND MOMENTS DUE TO PATTERN LOADING NO 2    FOR:
C                L.L./D.L. =0.25,0.333,0.5,1.0,2.0,3.0,4.0,3.643
C
C       P(62,5)=STRAINS AND MOMENTS DUE TO PATTERN LOADING NO 3    FOR:
C                L.L./D.L. =0.25,0.333,0.5,1.0,2.0,3.0,4.0,3.643
C
C       Q(62,5)=STRAINS AND MOMENTS DUE TO PATTERN LOADING NO 4    FOR:
C                L.L./D.L. =0.25,0.333,0.5,1.0,2,0,3.0,4.0,3.643
C
C       R(62,5)=STRAINS AND MOMENTS DUE TO PATTERN LOADING NO 5    FOR:
C                L.L./D.L. =0.25,0.333,0.5,1.0,2.0,3.0,4.0,3.643
C
C       S(62,5)=STRAINS AND MOMENTS DUE TO PATTERN LOADING NO 6    FOR:
C                L.L./D.L. =0.25,0.333,0.5,1.0,2.0,3.0,4,0,3.643
C
C
C     EXT(62,5)=EXTREME VALUES OF STRAINS AND MOMENTS FOR :
C                L.L./D.L. =0.0,0.25,0.333,0.5,1.0,2.0,3.0,4.0,3.643
C
C
      IMPLICIT REAL * 4(L,A,B,C,D,E,F,M,O,P,Q,R,S,T)
      INTEGER W
      DIMENSION L(62,4,9),A(62,3),B(62,3),C(62,3),D(62,3),E(62,3),F(62,3
     1),M(62,5),O(62,5),P(62,5),Q(62,5),R(62,5),S(62,5),T(62,3),EXT(62,5
     2)
      NR=5
      NW=6
      DO 10 K=1,9
      DO 10 I=1,62
   10 READ , (L(I,J,K),J=1,4)
      DO 20 I=1,62
      A(I,1)=L(I,2,1)
      DO 30 J=2,3
      A(I,J)=0.
      K=1
 1000 A(I,J)=A(I,J)+L(I,(J+1),K)+L(I,(J+1),(K+6))
      K=K+1
      IF(K.LE.3) GO TO 1000
   30 CONTINUE
   20 CONTINUE
      DO 40 I=1,62
      B(I,1)=L(I,2,1)
      DO 50 J=2,3
      B(I,J)=0.
      K=4
 1001 B(I,J)=B(I,J)+L(I,(J+1),K)
      K=K+1
      IF(K.LE.6) GO TO 1001
   50 CONTINUE
```

```
 40     CONTINUE
        DO 60 I=1,62
        C(I,1)=L(I,2,1)
        DO 70 J=2,3
        C(I,J)=0.
        K=1
 1002   C(I,J)=C(I,J)+L(I,(J+1),K)
 101    K=K+1
        IF(K.EQ.2) GO TO 101
        IF(K.EQ.5) GO TO 101
        IF(K.EQ.8) GO TO 101
        IF(K.LE.9) GO TO 1002
 70     CONTINUE
 60     CONTINUE
        DO 80 I=1,62
        D(I,1)=L(I,2,1)
        DO 90 J=2,3
        D(I,J)=0.
        K=2
 1003   D(I,J)=D(I,J)+L(I,(J+1),K)
        K=K+3
        IF(K.LE.8) GO TO 1003
 90     CONTINUE
 80     CONTINUE
        DO 110 I=1,62
        E(I,1)=L(I,2,1)
        DO 120 J=2,3
        E(I,J)=0.
        K=1
 1004   E(I,J)=E(I,J)+L(I,(J+1),K)
        K=K+1
        IF(K.LE.6) GO TO 1004
 120    CONTINUE
 110    CONTINUE
        DO 130 I=1,62
        F(I,1)=L(I,2,1)
        DO 140 J=2,3
        F(I,J)=0.
        K=1
 1005   F(I,J)=F(I,J)+L(I,(J+1),K)
 102    K=K+1
        IF (K.EQ.3) GO TO 102
        IF(K.EQ.6) GO TO 102
        IF(K.EQ.9) GO TO 102
        IF(K.LE.8) GO TO 1005
 140    CONTINUE
 130    CONTINUE
        DO 150 I=1,62
        T(I,1)=L(I,2,1)
        DO 160 J=2,3
        T(I,J)=0.
        K=1
```

```
 1006 T(I,J)=T(I,J)+L(I,(J+1),K)
      K=K+1
      IF (K.LE.9) GO TO 1006
  160 CONTINUE
  150 CONTINUE
      DO 300 K=1,9
      WRITE (NW,201)
  201 FORMAT ('1'/34X,68('*')/' ',33X,68('*')/' ',33X,68('*'))
      WRITE(NW,202)
  202 FORMAT(    34X,'***',62X,'***')
      WRITE(NW,202)
      WRITE(NW,203)K
  203 FORMAT(    34X,'***',5X,'STRAINS DUE TO     UNIFORM LOAD = 200 PSF O
     1N PANEL P',I1,5X,'***')
      WRITE(NW,202)
      WRITE(NW,204)K
  204 FORMAT(34X,'***',18X,'LOAD NO ',I1,6X,'FIRST LEVEL',18X,'***')
      WRITE(NW,202)
      WRITE(NW,202)
      WRITE(NW,205)
  205 FORMAT(    34X,68('*'))
      WRITE(NW,205)
      WRITE(NW,205)
      WRITE(NW,206)
  206 FORMAT(   ///   ,34X,68('*'))
      WRITE(NW,205)
      WRITE(NW,205)
      WRITE(NW,207)
  207 FORMAT(    34X,'***',6X,'*',11X,'*',11X,'**',6X,'*',11X,'*',11X,'**
     1*')
      WRITE(NW,208)
  208 FORMAT(    34X,'*** GAGE *   STRAINS   *   STRAINS   ** GAGE *   STRAINS
     1   *   STRAINS   ***')
      WRITE(NW,207)
      WRITE(NW,209)
  209 FORMAT(    34X,'***   NO   *    X-DIR.   *    Y-DIR.   **   NO   *    X-DIR.
     1   *    Y-DIR.   ***')
      WRITE(NW,207)
      WRITE(NW,210)
  210 FORMAT(    34X,'***        * IN X 10   * IN X 10   **        * IN X 10
     1   * IN X 10   ***')
      WRITE(NW,207)
      WRITE(NW,205)
      WRITE(NW,205)
      WRITE(NW,207)
      DO 310 I=1,31
      W=I+31
  310 WRITE(NW,211)(L(I,J,K),J=2,4),(L(W      ,J,K),J=2,4)
  211 FORMAT(    34X,'***',F6.1     ,'*',F9.1,2X,'*',F9.1,2X,'**',F6.1
     1,'*',F9.1,2X,'*',F9.1,2X,'***',T42,'   ',T74,'   '/34X,'***',6X,'*',
     211X,'*',11X,'**',6X,'*',11X,'*',11X,'***')
      WRITE(NW,205)
```

```
      WRITE(NW,205)
      WRITE(NW,205)
  300 CONTINUE
      WRITE(NW,201)
      WRITE(NW,202)
      WRITE(NW,202)
      WRITE(NW,212)
  212 FORMAT(   34X,'***',4X,'STRAINS DUE TO    UNIFORM LOAD = 200 PSF C
     1N ALL PANELS    ','***')
      WRITE(NW,202)
      WRITE(NW,213)
  213 FORMAT(   34X,'***',25X,'FIRST  LEVEL',25X,'***')
      WRITE(NW,202)
      WRITE(NW,202)
      WRITE(NW,205)
      WRITE(NW,205)
      WRITE(NW,205)
      WRITE(NW,206)
      WRITE(NW,205)
      WRITE(NW,205)
      WRITE(NW,207)
      WRITE(NW,208)
      WRITE(NW,207)
      WRITE(NW,209)
      WRITE(NW,207)
      WRITE(NW,210)
      WRITE(NW,207)
      WRITE(NW,205)
      WRITE(NW,205)
      WRITE(NW,207)
      DO 320  I=1,31
      W=I+31
  320 WRITE(NW,211)(T(I,J),J=1,3),(T(W     ,J),J=1,3)
      WRITE(NW,205)
      WRITE(NW,205)
      WRITE(NW,205)
      INO=1
      CALL TITLE (A,INO)
      INO=2
      CALL TITLE (B,INO)
      INO=3
      CALL TITLE (C,INO)
      INO=4
      CALL TITLE (D,INO)
      INO=5
      CALL TITLE (E,INO)
      INO=6
      CALL TITLE (F,INO)
      X=2.0
      Y=0.0
      CALL PATERN(A,B,C,D,E,F,T,M,O,P,Q,R,S,X,Y,EXT)
      X=1.6
```

```
      Y=.4
      CALL PATERN(A,B,C,D,E,F,T,M,O,P,Q,R,S,X,Y,EXT)
      X=1.5
      Y=.5
      CALL PATERN(A,B,C,D,E,F,T,M,O,P,Q,R,S,X,Y,EXT)
      X=1.3333
      Y=.6667
      CALL PATERN(A,B,C,D,E,F,T,M,O,P,Q,R,S,X,Y,EXT)
      X=1.
      Y=1.
      CALL PATERN(A,B,C,D,E,F,T,M,O,P,Q,R,S,X,Y,EXT)
      X=.6667
      Y=1.3333
      CALL PATERN(A,B,C,D,E,F,T,M,O,P,Q,R,S,X,Y,EXT)
      X=.5
      Y=1.5
      CALL PATERN(A,B,C,D,E,F,T,M,O,P,Q,R,S,X,Y,EXT)
      X=.431
      Y=1.569
      CALL PATERN(A,B,C,D,E,F,T,M,O,P,Q,R,S,X,Y,EXT)
      X=.4
      Y=1.6
      CALL PATERN(A,B,C,D,E,F,T,M,O,P,Q,R,S,X,Y,EXT)
      STOP
      END
C
C
C
C
      SUBROUTINE TITLE(A,INO)
      DIMENSION A(62,3)
      NW=6
      WRITE (NW,214)INO
  214 FORMAT('1'/34X,68('*')/   34X,68('*')/   34X,68('*'),2(/   34X,'**
     1*',62X,'***')/   34X,'***',3X,'STRAINS DUE TO   UNIFORM L.L. = 200
     2 PSF ON MARKED PANELS     ***'/   34X,'***',62X,'***'/   34X,'***',1
     30X,'L.L. PATTERN LOADING NO.',I2,4X,'FIRST  LEVEL',10X,'***',2(/34
     4X,'***',62X,'***'),3(/34X,68('*'))//3(/34X,68('*')))
      WRITE(NW,207)
      WRITE(NW,208)
      WRITE(NW,207)
      WRITE(NW,209)
      WRITE(NW,207)
      WRITE(NW,210)
      WRITE(NW,207)
      WRITE(NW,205)
      WRITE(NW,205)
      WRITE(NW,207)
  205 FORMAT(   34X,68('*'))
  207 FORMAT(   34X,'***',6X,'*',11X,'*',11X,'**',6X,'*',11X,'*',11X,'**
     1*')
```

```
  208 FORMAT(   34X,'*** GAGE *  STRAINS  *  STRAINS  ** GAGE *  STRAIN
     1  *  STRAINS  ***')
  209 FORMAT(   34X,'***  NO  *   X-DIR.  *   Y-DIR.  **  NO  *   X-DIR.
     1  *   Y-DIR.  ***')
  210 FORMAT(   34X,'***        * IN X 10   * IN X 10   **       * IN X 10
     1  * IN X 10   ***')
      DO 330 I=1,31
  330 WRITE(NW,215)(A(I,J),J=1,3),(A((I+31),J),J=1,3)
  215 FORMAT(   34X,'***',F6.1    ,'*',F9.1,2X,'*',F9.1,2X,'**',F6.1
     1,'*',F9.1,2X,'*',F9.1,2X,'***',T42,'  ',T74,'  '/34X,'***',6X,'*',
     211X,'*',11X,'**',6X,'*',11X,'*',11X,'***')
      WRITE(NW,205)
      WRITE(NW,205)
      WRITE(NW,205)
      RETURN
      END
C
C
C
C
      SUBROUTINE PATERN(A,B,C,D,E,F,T,M,O,P,Q,R,S,X,Y,EXT)
      IMPLICIT REAL*4(M)
      INTEGER W
      DIMENSION A(62,3),B(62,3),C(62,3),D(62,3),E(62,3),F(62,3),T(62,3),
     1M(62,5),O(62,5),P(62,5),Q(62,5),R(62,5),S(62,5),EXT(62,5)
      W=1
      CALL MOMENT(A,T,M,X,Y,W)
      W=2
      CALL MOMENT(B,T,O,X,Y,W)
      W=3
      CALL MOMENT(C,T,P,X,Y,W)
      W=4
      CALL MOMENT(D,T,Q,X,Y,W)
      W=5
      CALL MOMENT(E,T,R,X,Y,W)
      W=6
      CALL MOMENT(F,T,S,X,Y,W)
      CALL EXTREM(M,O,P,Q,R,S,X,Y,EXT)
      RETURN
      END
C
C
C
C
      SUBROUTINE EXTREM(M,O,P,Q,R,S,X,Y,EXT)
      IMPLICIT REAL*4(M)
      DIMENSION M(62,5),O(62,5),P(62,5),Q(62,5),R(62,5),S(62,5),EXT(62,
     15)
      NW=6
      DO 700 I=1,62
```

```
  700 EXT(I,1)=M(I,1)
      DO 710 J=2,4,2
      DO 720 I=1,28
      EXT(I,J)=P(I,J)
  720 CONTINUE
      DO 730 I=29,45
      EXT(I,J)=S(I,J)
  730 CONTINUE
      DO 740 I=46,56
      EXT(I,J)=O(I,J)
  740 CONTINUE
      EXT(57,J)=P(57,J)
      EXT(58,J)=S(58,J)
      EXT(59,J)=S(59,J)
      EXT(60,J)=P(60,J)
      EXT(61,J)=S(61,J)
      EXT(62,J)=S(62,J)
  710 CONTINUE
      DO 750 J=3,5,2
      EXT(1,J)=O(1,J)
      EXT(2,J)=O(2,J)
      EXT(3,J)=R(3,J)
      EXT(4,J)=R(4,J)
      EXT(5,J)=M(5,J)
      EXT(6,J)=M(6,J)
      EXT(7,J)=M(7,J)
      EXT(8,J)=M(8,J)
      EXT( 9,J)=M( 9,J)
      EXT(10,J)=O(10,J)
      EXT(11,J)=P(11,J)
      EXT(12,J)=M(12,J)
      EXT(13,J)=M(13,J)
      EXT(14,J)=O(14,J)
      EXT(15,J)=P(15,J)
      EXT(16,J)=M(16,J)
      EXT(17,J)=M(17,J)
      EXT(18,J)=O(18,J)
      EXT(19,J)=O(19,J)
      EXT(20,J)=R(20,J)
      EXT(21,J)=M(21,J)
      EXT(22,J)=M(22,J)
      EXT(23,J)=M(23,J)
      EXT(24,J)=M(24,J)
      EXT(25,J)=O(25,J)
      EXT(26,J)=R(26,J)
      EXT(27,J)=M(27,J)
      EXT(28,J)=M(28,J)
      EXT(29,J)=O(29,J)
      EXT(30,J)=R(30,J)
      EXT(31,J)=M(31,J)
      EXT(32,J)=M(32,J)
      EXT(33,J)=O(33,J)
```

```
      EXT(34,J)=O(34,J)
      EXT(35,J)=R(35,J)
      EXT(36,J)=R(36,J)
      EXT(37,J)=M(37,J)
      EXT(38,J)=M(38,J)
      EXT(39,J)=M(39,J)
      EXT(40,J)=M(40,J)
      EXT(41,J)=M(41,J)
      EXT(42,J)=O(42,J)
      EXT(43,J)=R(43,J)
      EXT(44,J)=M(44,J)
      EXT(45,J)=M(45,J)
      EXT(46,J)=O(46,J)
      EXT(47,J)=R(47,J)
      EXT(48,J)=M(48,J)
      EXT(49,J)=M(49,J)
      EXT(50,J)=O(50,J)
      EXT(51,J)=O(51,J)
      EXT(52,J)=R(52,J)
      EXT(53,J)=M(53,J)
      EXT(54,J)=M(54,J)
      EXT(55,J)=M(55,J)
      EXT(56,J)=M(56,J)
      EXT(57,J)=R(57,J)
      EXT(58,J)=R(58,J)
      EXT(59,J)=R(59,J)
      EXT(60,J)=M(60,J)
      EXT(61,J)=M(61,J)
      EXT(62,J)=M(62,J)
  750 CONTINUE
      WRITE(NW,449)
  449 FORMAT('1'//19X,100('*'))
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,500)
  500 FORMAT(19X,100('*'))
      WRITE(NW,501)
      WRITE(NW,501)
  501 FORMAT(19X,'***',94X,'***')
      WRITE(NW,701)
  701 FORMAT(19X,'***',24X,'EXTREME VALUES OF   STRAINS AND MOMENTS DUE
     1TO',24X,'***')
      WRITE(NW,501)
      G=200.*X
      H=200.*Y
      WRITE(NW,702)G,H
  702 FORMAT(19X,'***',21X,'UNIFORM D.L.=',F6.1,' PSF   + UNIFORM L.L.=
     1',F6.1,' PSF',22X,'***')
      WRITE(NW,501)
      WRITE(NW,703)
  703 FORMAT(19X,'***',41X,'FIRST  LEVEL',41X,'***')
      WRITE(NW,501)
```

```
      WRITE(NW,501)
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,704)
  704 FORMAT(//19X,100('*'))
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,506)
  506 FORMAT(19X,'**',2('*',6X,'*',9X,'*',9X,'*',9X,'*',9X,'*'),'**')
      WRITE(NW,507)
  507 FORMAT(19X,'**',2('* GAGE * STRAINS * STRAINS * MOMENTS * MOMENTS
     1*'),'**')
      WRITE(NW,506)
      WRITE(NW,508)
  508 FORMAT(19X,'**',2('*  NO   *  X-DIR. *  Y-DIR. *  X-DIR. *  Y-DIR.
     1*'),'**')
      WRITE(NW,506)
      WRITE(NW,509)
  509 FORMAT(19X,'**',2('*       * INX10   * INX10   * LB. IN. * LB. IN.
     1*'),'**')
      WRITE(NW,506)
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,506)
      DO 360 I=1,31
  360 WRITE(NW,510)(EXT(I,J),J=1,5),(EXT((I+31),J),J=1,5)
  510 FORMAT(19X,'***',F6.1,'*',F8.1,1X,'*',F8.1,1X,'*',F8.1,1X,'*',F8.1
     1,1X,'**',F6.1,'*',F8.1,1X,'*',F8.1,1X,'*',F8.1,1X,'*',F8.1,1X,'***
     2',T27,'  ',T75,'  '/19X,'**',2('*',6X,'*',9X,'*',9X,'*',9X,'*',9X,
     3'*'),'**')
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,500)
      RETURN
      END
C
C
C
C
      SUBROUTINE MOMENT(A,T,M,X,Y,W)
      IMPLICIT REAL * 4(A,T,M,X,Y)
      INTEGER W
      DIMENSION A(62,3),T(62,3),M(62,5)
      NW=6
      U=20949.2
      V=7332.2
      DO 340 I=1,62
      M(I,1)=A(I,1)
      DO 350 J=2,3
```

```
      M(I,J)=X*T(I,J)+Y*A(I,J)
  350 CONTINUE
      M(I,4)=(U*M(I,2)+V*M(I,3))/1000000.0
      M(I,5)=(U*M(I,3)+V*M(I,2))/1000000.0
  340 CONTINUE
      IF(X.EQ.2.)GO TO 800
      WRITE(NW,449)
  449 FORMAT('1'///19X,100('*'))
      WRITE(NW,500)
      WRITE(NW,500)
  500 FORMAT(19X,100('*'))
      WRITE(NW,501)
      WRITE(NW,501)
  501 FORMAT(19X,'***',94X,'***')
      WRITE(NW,502)
  502 FORMAT(19X,'***',34X,'STRAINS AND MOMENTS DUE TO',34X,'***')
      WRITE(NW,501)
      G=200.*X
      H=200.*Y
      WRITE(NW,503)G,H
  503 FORMAT(19X,'***',13X,'UNIFORM D.L.=',F6.1,' PSF    + UNIFORM L.L.='
     1,F6.1,' PSF ON MARKED PANELS',13X,'***')
      WRITE(NW,501)
      WRITE(NW,504)W
  504 FORMAT(19X,'***',29X,'PATTERN LOADING  NO.',I2,'    FIRST LEVEL',2
     1X,'***')
      WRITE(NW,501)
      WRITE(NW,501)
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,505)
  505 FORMAT(///19X,100('*'))
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,506)
  506 FORMAT(19X,'**',2('*',6X,'*',9X,'*',9X,'*',9X,'*',9X,'*'),'**')
      WRITE(NW,507)
  507 FORMAT(19X,'**',2('* GAGE * STRAINS * STRAINS * MOMENTS * MOMENTS
     1*'),'**')
      WRITE(NW,506)
      WRITE(NW,508)
  508 FORMAT(19X,'**',2('*  NO  *  X-DIR. *  Y-DIR. *  X-DIR. *  Y-DIR.
     1*'),'**')
      WRITE(NW,506)
      WRITE(NW,509)
  509 FORMAT(19X,'**',2('*      * INX10   * INX10   * LB. IN. * LB. IN.
     1*'),'**')
      WRITE(NW,506)
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,506)
```

```
      DO 360 I=1,31
360   WRITE(NW,510) (M(I,J),J=1,5),(M((I+31),J),J=1,5)
510   FORMAT(19X,'***',F6.1,'*',F8.1,1X,'*',F8.1,1X,'*',F8.1,1X,'*',F8.1
     1,1X,'**',F6.1,'*',F8.1,1X,'*',F8.1,1X,'*',F8.1,1X,'*',F8.1,1X,'***
     2',T27,'  ',T75,'   '/19X,'**',2('*',6X,'*',9X,'*',9X,'*',9X,'*',9X,
     3'*'),'**')
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,500)
800   RETURN
      END

$ENTRY
```

```
$JOB    SK411310,FALTACUS,PAGES=80,TIME=120,LINES=12000
C
C
C       ANALYSIS OF FLAT PLATES
C
C       EXPERIMENTAL INVESTIGATION
C
C
C       THIS PROGRAM USES EXPERIMENTALLY MEASURED STRAINS FOR
C       THE CALCULATIONS OF MOMENTS IN THE 'X' AND 'Y' DIRECTIONS
C       FOR DIFFERENT LOADING CASES
C
C
C                 MODEL M1
C                ==========
C
C                 SECOND LEVEL
C                ==============
C
C
C       EXPLANATION OF SYMBOLS USED
C       ----------------------------
C
C                  FIRST COLUMN REPRESENTS THE LOAD NUMBER,
C                  SECOND COLUMN REPRESENTS STRAIN GAGE NUMBER,
C                  THIRD AND FORTH COLUMNS ARE THE STRAINS IN THE 'X' AND
C                  'Y' DIRECTIONS
C
C       A(62,3)   =STRAINS DUE TO PATTERN LOADING NO'1'
C                  (L.L.=200 PSF ON PANELS P1,P2,P3,P7,P8,P9)
C
C       B(62,3)   =STRAINS DUE TO PATTERN LOADING NO'2'
C                  (L.L.=200 PSF ON PANELS P2,P3,P4)
C
C       C(62,3)   =STRAINS DUE TO PATTERN LOADING NO'3'
C                  (L.L.=200 PSF ON PANEL P1,P3,P4,P6,P7,P9)
C
C       D(62,3)   =STRAINS DUE TO PATTERN LOADING NO'4'
C                  (L.L.=200 PSF ON PANELS P2,P5,P8)
C
C       E(62,3)   =STRAINS DUE TO PATTERN LOADING NO'5'
C                  (L.L.=200 PSF ON PANELS P1,P2,P3,P4,P5,P6)
C
C       F(62,3)   =STRAINS DUE TO PATTERN LOADING NO'6'
C                  (L.L.=200 PSF ON PANELS P1,P2,P4,P5,P7,P8)
C
C
C       T(62,3)   =STRAINS DUE TO UNIFORM LOAD=200 PSF ON ALL PANELS
C
C
C       M(62,5)=STRAINS AND MOMENTS DUE TO PATTERN LOADING NO 1    FOR:
C               L.L./D.L. =0.25,0.333,0.5,1.0,2.0,3.0,4.0,3.643
```

```
C
C      O(62,5)=STRAINS AND MOMENTS DUE TO PATTERN LOADING NO 2    FOR:
C              L.L./D.L. =0.25,0.333,0.5,1.0,2.0,3.0,4.0,3.643
C
C      P(62,5)=STRAINS AND MOMENTS DUE TO PATTERN LOADING NO 3    FOR:
C              L.L./D.L. =0.25,0.333,0.5,1.0,2.0,3.0,4.0,3.643
C
C      Q(62,5)=STRAINS AND MOMENTS DUE TO PATTERN LOADING NO 4    FOR:
C              L.L./D.L. =0.25,0.333,0.5,1.0,2.0,3.0,4.0,3.643
C
C      R(62,5)=STRAINS AND MOMENTS DUE TO PATTERN LOADING NO 5    FOR:
C              L.L./D.L. =0.25,0.333,0.5,1.0,2.0,3.0,4.0,3.643
C
C      S(62,5)=STRAINS AND MOMENTS DUE TO PATTERN LOADING NO 6    FOR:
C              L.L./D.L. =0.25,0.333,0.5,1.0,2.0,3.0,4.0,3.643
C
C
C
C    EXT(62,5)=EXTREME VALUES OF STRAINS AND MOMENTS FOR :
C              L.L./D.L. =0.0,0.25,0.333,0.5,1.0,2.0,3.0,4.0,3.643
C
C
      IMPLICIT REAL * 4(L,A,B,C,D,E,F,M,O,P,Q,R,S,T)
      INTEGER W,Z
      DIMENSION L(62,4,9),A(62,3),B(62,3),C(62,3),D(62,3),E(62,3),F(62,3
     1),M(62,5),O(62,5),P(62,5),Q(62,5),R(62,5),S(62,5),T(62,3),EXT(62,5
     2)
      NR=5
      NW=6
      DO 10 K=1,9
      DO 10 I=1,62
   10 READ , (L(I,J,K),J=1,4)
      DO 20 I=1,62
      A(I,1)=L(I,2,1)
      DO 30 J=2,3
      A(I,J)=0.
      K=1
 1000 A(I,J)=A(I,J)+L(I,(J+1),K)+L(I,(J+1),(K+5))
      K=K+1
      IF(K.LE.3) GO TO 1000
   30 CONTINUE
   20 CONTINUE
      DO 40 I=1,62
      B(I,1)=L(I,2,1)
      DO 50 J=2,3
      B(I,J)=0.
      K=4
 1001 B(I,J)=B(I,J)+L(I,(J+1),K)
      K=K+1
      IF(K.LE.6) GO TO 1001
   50 CONTINUE
   40 CONTINUE
```

```
      DO 60 I=1,62
      C(I,1)=L(I,2,1)
      DO 70 J=2,3
      C(I,J)=0.
      K=1
1002  C(I,J)=C(I,J)+L(I,(J+1),K)
101   K=K+1
      IF(K.EQ.2) GO TO 101
      IF(K.EQ.5) GO TO 101
      IF(K.EQ.8) GO TO 101
      IF(K.LE.9) GO TO 1002
70    CONTINUE
60    CONTINUE
      DO 80 I=1,62
      D(I,1)=L(I,2,1)
      DO 90 J=2,3
      D(I,J)=0.
      K=2
1003  D(I,J)=D(I,J)+L(I,(J+1),K)
      K=K+3
      IF(K.LE.8) GO TO 1003
90    CONTINUE
80    CONTINUE
      DO 110 I=1,62
      E(I,1)=L(I,2,1)
      DO 120 J=2,3
      E(I,J)=0.
      K=1
1004  E(I,J)=E(I,J)+L(I,(J+1),K)
      K=K+1
      IF(K.LE.6) GO TO 1004
120   CONTINUE
110   CONTINUE
      DO 130 I=1,62
      F(I,1)=L(I,2,1)
      DO 140 J=2,3
      F(I,J)=0.
      K=1
1005  F(I,J)=F(I,J)+L(I,(J+1),K)
102   K=K+1
      IF (K.EQ.3) GO TO 102
      IF(K.EQ.6) GO TO 102
      IF(K.EQ.9) GO TO 102
      IF(K.LE.8) GO TO 1005
140   CONTINUE
130   CONTINUE
      DO 150 I=1,62
      T(I,1)=L(I,2,1)
      DO 160 J=2,3
      T(I,J)=0.
      K=1
1006  T(I,J)=T(I,J)+L(I,(J+1),K)
```

```
          K=K+1
          IF (K.LE.9) GO TO 1006
    160   CONTINUE
    150   CONTINUE
          DO 300 K=1,9
          Z=K+9
          WRITE (NW,201)
    201   FORMAT ('1'/34X,68('*')/' ',33X,68('*')/' ',33X,68('*'))
          WRITE(NW,202)
    202   FORMAT(    34X,'***',62X,'***')
          WRITE(NW,202)
          WRITE(NW,203)K
    203   FORMAT(    34X,'***',5X,'STRAINS DUE TO     UNIFORM LOAD = 200 PSF O
         1N PANEL P',I1,5X,'***')
          WRITE(NW,202)
          WRITE(NW,204)Z
    204   FORMAT(34X,'***',17X,'LOAD NO ',I2,6X,'SECOND LEVEL',17X,'***')
          WRITE(NW,202)
          WRITE(NW,202)
          WRITE(NW,205)
    205   FORMAT(    34X,68('*'))
          WRITE(NW,205)
          WRITE(NW,205)
          WRITE(NW,206)
    206   FORMAT(  ///   ,34X,68('*'))
          WRITE(NW,205)
          WRITE(NW,205)
          WRITE(NW,207)
    207   FORMAT(    34X,'***',6X,'*',11X,'*',11X,'**',6X,'*',11X,'*',11X,'**
         1*')
          WRITE(NW,208)
    208   FORMAT(    34X,'*** GAGE *  STRAINS  *  STRAINS  ** GAGE *  STRAINS
         1  *  STRAINS  ***')
          WRITE(NW,207)
          WRITE(NW,209)
    209   FORMAT(    34X,'***  NO  *   X-DIR.  *   Y-DIR.  **  NO  *   X-DIR.
         1  *   Y-DIR.  ***')
          WRITE(NW,207)
          WRITE(NW,210)
    210   FORMAT(    34X,'***      * IN X 10  * IN X 10  **      * IN X 10
         1  * IN X 10  ***')
          WRITE(NW,207)
          WRITE(NW,205)
          WRITE(NW,205)
          WRITE(NW,207)
          DO 310  I=1,31
          W=I+31
    310   WRITE(NW,211)(L(I,J,K),J=2,4),(L(W      ,J,K),J=2,4)
    211   FORMAT(    34X,'***',F6.1     ,'*',F9.1,2X,'*',F9.1,2X,'**',F6.1
         1,'*',F9.1,2X,'*',F9.1,2X,'***',T42,'   ',T74,'   '/34X,'***',6X,'*',
         211X,'*',11X,'**',6X,'*',11X,'*',11X,'***')
          WRITE(NW,205)
```

```
          WRITE(NW,205)
          WRITE(NW,205)
    300   CONTINUE
          WRITE(NW,201)
          WRITE(NW,202)
          WRITE(NW,202)
          WRITE(NW,212)
    212   FORMAT(    34X,'***',4X,'STRAINS DUE TO     UNIFORM LOAD = 200 PSF O
         1N ALL PANELS      ','***')
          WRITE(NW,202)
          WRITE(NW,213)
    213   FORMAT(    34X,'***',24X,'SECOND  LEVEL',24X,'***')
          WRITE(NW,202)
          WRITE(NW,202)
          WRITE(NW,205)
          WRITE(NW,205)
          WRITE(NW,205)
          WRITE(NW,206)
          WRITE(NW,205)
          WRITE(NW,205)
          WRITE(NW,207)
          WRITE(NW,208)
          WRITE(NW,207)
          WRITE(NW,209)
          WRITE(NW,207)
          WRITE(NW,210)
          WRITE(NW,207)
          WRITE(NW,205)
          WRITE(NW,205)
          WRITE(NW,207)
          DO 320   I=1,31
          W=I+31
    320   WRITE(NW,211)(T(I,J),J=1,3),(T(W      ,J),J=1,3)
          WRITE(NW,205)
          WRITE(NW,205)
          WRITE(NW,205)
          INO=1
          CALL TITLE (A,INO)
          INO=2
          CALL TITLE (B,INO)
          INO=3
          CALL TITLE (C,INO)
          INO= 4
          CALL TITLE (D,INO)
          INO=5
          CALL TITLE (E,INO)
          INO=6
          CALL TITLE (F,INO)
          X=2.0
          Y=0.0
          CALL PATERN(A,B,C,D,E,F,T,M,O,P,Q,R,S,X,Y,EXT)
          X=1.6
```

```
      Y=.4
      CALL PATERN(A,B,C,D,E,F,T,M,O,P,Q,R,S,X,Y,EXT)
      X=1.5
      Y=.5
      CALL PATERN(A,B,C,D,E,F,T,M,O,P,Q,R,S,X,Y,EXT)
      X=1.3333
      Y=.6667
      CALL PATERN(A,B,C,D,E,F,T,M,O,P,Q,R,S,X,Y,EXT)
      X=1.
      Y=1.
      CALL PATERN(A,B,C,D,E,F,T,M,O,P,Q,R,S,X,Y,EXT)
      X=.6667
      Y=1.3333
      CALL PATERN(A,B,C,D,E,F,T,M,O,P,Q,R,S,X,Y,EXT)
      X=.5
      Y=1.5
      CALL PATERN(A,B,C,D,E,F,T,M,O,P,Q,R,S,X,Y,EXT)
      X=.431
      Y=1.569
      CALL PATERN(A,B,C,D,E,F,T,M,O,P,Q,R,S,X,Y,EXT)
      X=.4
      Y=1.6
      CALL PATERN(A,B,C,D,E,F,T,M,O,P,Q,R,S,X,Y,EXT)
      STOP
      END
C
C
C
C

      SUBROUTINE TITLE(A,INO)
      DIMENSION A(62,3)
      NW=6
      WRITE (NW,214)INO
  214 FORMAT('1'/34X,68('*')/    34X,68('*')/    34X,68('*'),2(/    34X,'**
     1*',62X,'***')/    34X,'***',3X,'STRAINS DUE TO   UNIFORM L.L. = 200
     2 PSF ON MARKED PANELS    ***'/    34X,'***',62X,'***'/    34X,'***',1
     30X,'L.L. PATTERN LOADING NO.',I2,4X,'SECOND LEVEL',10X,'***',2(/34
     4X,'***',62X,'***'),3(/34X,68('*'))//3(/34X,68('*')))
      WRITE(NW,207)
      WRITE(NW,208)
      WRITE(NW,207)
      WRITE(NW,209)
      WRITE(NW,207)
      WRITE(NW,210)
      WRITE(NW,207)
      WRITE(NW,205)
      WRITE(NW,205)
      WRITE(NW,207)
  205 FORMAT(    34X,68('*'))
  207 FORMAT(    34X,'***',6X,'*',11X,'*',11X,'**',6X,'*',11X,'*',11X,'**
     1*')
```

```
  208 FORMAT(    34X,'*** GAGE *  STRAINS  *  STRAINS  ** GAGE *  STRAINS
     1  *  STRAINS  ***')
  209 FORMAT(    34X,'***  NO  *   X-DIR.  *   Y-DIR.  **  NO  *   X-DIR.
     1  *   Y-DIR.  ***')
  210 FORMAT(    34X,'***      * IN X 10   * IN X 10   **      * IN X 10
     1  * IN X 10   ***')
      DO 330 I=1,31
  330 WRITE(NW,215)(A(I,J),J=1,3),(A((I+31),J),J=1,3)
  215 FORMAT(    34X,'***',F6.1    ,'*',F9.1,2X,'*',F9.1,2X,'**',F6.1
     1,'*',F9.1,2X,'*',F9.1,2X,'***',T42,'  ',T74,'  '/34X,'***',6X,'*',
     211X,'*',11X,'**',6X,'*',11X,'*',11X,'***')
      WRITE(NW,205)
      WRITE(NW,205)
      WRITE(NW,205)
      RETURN
      END
C
C
C
C
      SUBROUTINE PATERN(A,B,C,D,E,F,T,M,O,P,Q,R,S,X,Y,EXT)
      IMPLICIT REAL*4(M)
      INTEGER W
      DIMENSION A(62,3),B(62,3),C(62,3),D(62,3),E(62,3),F(62,3),T(62,3),
     1M(62,5),O(62,5),P(62,5),Q(62,5),R(62,5),S(62,5),EXT(62,5)
      W=1
      CALL MOMENT(A,T,M,X,Y,W)
      W=2
      CALL MOMENT(B,T,O,X,Y,W)
      W=3
      CALL MOMENT(C,T,P,X,Y,W)
      W=4
      CALL MOMENT(D,T,Q,X,Y,W)
      W=5
      CALL MOMENT(E,T,R,X,Y,W)
      W=6
      CALL MOMENT(F,T,S,X,Y,W)
      CALL EXTREM(M,O,P,Q,R,S,X,Y,EXT)
      RETURN
      END
C
C
C
C
      SUBROUTINE EXTREM(M,O,P,Q,R,S,X,Y,EXT)
      IMPLICIT REAL*4(M)
      DIMENSION M(62,5),O(62,5),P(62,5),Q(62,5),R(62,5),S(62,5),EXT(62,
     15)
      NW=6
      DO 700 I=1,62
```

```
 700  EXT(I,1)=M(I,1)
      DO 710 J=2,4,2
      DO 720 I=1,28
      EXT(I,J)=P(I,J)
 720  CONTINUE
      DO 730 I=29,45
      EXT(I,J)=S(I,J)
 730  CONTINUE
      DO 740 I=46,56
      EXT(I,J)=Q(I,J)
 740  CONTINUE
      EXT(57,J)=P(57,J)
      EXT(58,J)=S(58,J)
      EXT(59,J)=S(59,J)
      EXT(60,J)=P(60,J)
      EXT(61,J)=S(61,J)
      EXT(62,J)=S(62,J)
 710  CONTINUE
      DO 750 J=3,5,2
      EXT(1,J)=O(1,J)
      EXT(2,J)=O(2,J)
      EXT(3,J)=R(3,J)
      EXT(4,J)=R(4,J)
      EXT(5,J)=M(5,J)
      EXT(6,J)=M(6,J)
      EXT(7,J)=M(7,J)
      EXT(8,J)=M(8,J)
      EXT( 9,J)=M( 9,J)
      EXT(10,J)=O(10,J)
      EXT(11,J)=P(11,J)
      EXT(12,J)=M(12,J)
      EXT(13,J)=M(13,J)
      EXT(14,J)=O(14,J)
      EXT(15,J)=R(15,J)
      EXT(16,J)=M(16,J)
      EXT(17,J)=M(17,J)
      EXT(18,J)=O(18,J)
      EXT(19,J)=O(19,J)
      EXT(20,J)=R(20,J)
      EXT(21,J)=M(21,J)
      EXT(22,J)=M(22,J)
      EXT(23,J)=M(23,J)
      EXT(24,J)=M(24,J)
      EXT(25,J)=O(25,J)
      EXT(26,J)=R(26,J)
      EXT(27,J)=M(27,J)
      EXT(28,J)=M(28,J)
      EXT(29,J)=O(29,J)
      EXT(30,J)=R(30,J)
      EXT(31,J)=M(31,J)
      EXT(32,J)=M(32,J)
      EXT(33,J)=O(33,J)
```

```
      EXT(34,J)=O(34,J)
      EXT(35,J)=R(35,J)
      EXT(36,J)=R(36,J)
      EXT(37,J)=M(37,J)
      EXT(38,J)=M(38,J)
      EXT(39,J)=M(39,J)
      EXT(40,J)=M(40,J)
      EXT(41,J)=M(41,J)
      EXT(42,J)=O(42,J)
      EXT(43,J)=R(43,J)
      EXT(44,J)=M(44,J)
      EXT(45,J)=M(45,J)
      EXT(46,J)=O(46,J)
      EXT(47,J)=R(47,J)
      EXT(48,J)=M(48,J)
      EXT(49,J)=M(49,J)
      EXT(50,J)=O(50,J)
      EXT(51,J)=O(51,J)
      EXT(52,J)=R(52,J)
      EXT(53,J)=M(53,J)
      EXT(54,J)=M(54,J)
      EXT(55,J)=M(55,J)
      EXT(56,J)=M(56,J)
      EXT(57,J)=R(57,J)
      EXT(58,J)=R(58,J)
      EXT(59,J)=R(59,J)
      EXT(60,J)=M(60,J)
      EXT(61,J)=M(61,J)
      EXT(62,J)=M(62,J)
  750 CONTINUE
      WRITE(NW,449)
  449 FORMAT('1'//19X,100('*'))
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,500)
  500 FORMAT(19X,100('*'))
      WRITE(NW,501)
      WRITE(NW,501)
  501 FORMAT(19X,'***',94X,'***')
      WRITE(NW,701)
  701 FORMAT(19X,'***',24X,'EXTREME VALUES OF   STRAINS AND MOMENTS DUE
     1TO',24X,'***')
      WRITE(NW,501)
      G=200.*X
      H=200.*Y
      WRITE(NW,702)G,H
  702 FORMAT(19X,'***',21X,'UNIFORM D.L.=',F6.1,' PSF   + UNIFORM L.L.='
     1,F6.1,' PSF',22X,'***')
      WPITE(NW,501)
      WRITE(NW,703)
  703 FORMAT(19X,'***',41X,'SECOND LEVEL',41X,'***')
      WRITE(NW,501)
```

```
      WRITE(NW,501)
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,704)
  704 FORMAT(//19X,100('*'))
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,506)
  506 FORMAT(19X,'**',2('*',6X,'*',9X,'*',9X,'*',9X,'*',9X,'*'),'**')
      WRITE(NW,507)
  507 FORMAT(19X,'**',2('* GAGE * STRAINS * STRAINS * MOMENTS * MOMENTS
     1*'),'**')
      WRITE(NW,506)
      WRITE(NW,508)
  508 FORMAT(19X,'**',2('*  NO   *  X-DIR. *  Y-DIR. *  X-DIR. *  Y-DIR.
     1*'),'**')
      WRITE(NW,506)
      WRITE(NW,509)
  509 FORMAT(19X,'**',2('*       * INX10   * INX10   * LB. IN. * LB. IN.
     1*'),'**')
      WRITE(NW,506)
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,506)
      DO 360 I=1,31
  360 WRITE(NW,510)(EXT(I,J),J=1,5),(EXT((I+31),J),J=1,5)
  510 FORMAT(19X,'***',F6.1,'*',F8.1,1X,'*',F8.1,1X,'*',F8.1,1X,'*',F8.1
     1,1X,'**',F6.1,'*',F8.1,1X,'*',F8.1,1X,'*',F8.1,1X,'*',F8.1,1X,'***
     2',T27,'  ',T75,'  '/19X,'**',2('*',6X,'*',9X,'*',9X,'*',9X,'*',9X,
     3'*'),'**')
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,500)
      RETURN
      END
C
C
C
C
      SUBROUTINE MOMENT(A,T,M,X,Y,W)
      IMPLICIT REAL * 4(A,T,M,X,Y)
      INTEGER W
      DIMENSION A(62,3),T(62,3),M(62,5)
      NW=6
      U=20949.2
      V=7332.2
      DO 340 I=1,62
      M(I,1)=A(I,1)
      DO 350 J=2,3
```

```
      M(I,J)=X*T(I,J)+Y*A(I,J)
  350 CONTINUE
      M(I,4)=(U*M(I,2)+V*M(I,3))/1000000.0
      M(I,5)=(U*M(I,3)+V*M(I,2))/1000000.0
  340 CONTINUE
      IF(X.EQ.2.)GO TO 800
      WRITE(NW,449)
  449 FORMAT('1'///19X,100('*'))
      WRITE(NW,500)
      WRITE(NW,500)
  500 FORMAT(19X,100('*'))
      WRITE(NW,501)
      WRITE(NW,501)
  501 FORMAT(19X,'***',94X,'***')
      WRITE(NW,502)
  502 FORMAT(19X,'***',34X,'STRAINS AND MOMENTS DUE TO',34X,'***')
      WRITE(NW,501)
      G=200.*X
      H=200.*Y
      WRITE(NW,503)G,H
  503 FORMAT(19X,'***',13X,'UNIFORM D.L.=',F6.1,' PSF    + UNIFORM L.L.='
     1,F6.1,' PSF ON MARKED PANELS',13X,'***')
      WRITE(NW,501)
      WRITE(NW,504)W
  504 FORMAT(19X,'***',29X,'PATTERN LOADING  NO.',I2,'    SECOND LEVEL',2
     18X,'***')
      WRITE(NW,501)
      WRITE(NW,501)
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,505)
  505 FORMAT(///19X,100('*'))
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,506)
  506 FORMAT(19X,'**',2('*',6X,'*',9X,'*',9X,'*',9X,'*',9X,'*'),'**')
      WRITE(NW,507)
  507 FORMAT(19X,'**',2('* GAGE * STRAINS * STRAINS * MOMENTS * MOMENTS
     1*'),'**')
      WRITE(NW,506)
      WRITE(NW,508)
  508 FORMAT(19X,'**',2('*  NO   *  X-DIR. *  Y-DIR. *  X-DIR. *  Y-DIR.
     1*'),'**')
      WRITE(NW,506)
      WRITE(NW,509)
  509 FORMAT(19X,'**',2('*       * INX10    * INX10    * LB. IN. * LB. IN.
     1*'),'**')
      WRITE(NW,506)
      WRITE(NW,500)
      WRITE(NW,500)
      WRITE(NW,506)
```

```
          DO 360 I=1,31
    360   WRITE(NW,510) (M(I,J),J=1,5),(M((I+31),J),J=1,5)
    510   FORMAT(19X,'***',F6.1,'*',F8.1,1X,'*',F8.1,1X,'*',F8.1,1X,'*',F8.1
         1,1X,'**',F6.1,'*',F8.1,1X,'*',F8.1,1X,'*',F8.1,1X,'*',F8.1,1X,'***
         2',T27,'  ',T75,'  '/19X,'**',2('*',6X,'*',9X,'*',9X,'*',9X,'*',9X,
         3'*'),'**')
          WRITE(NW,500)
          WRITE(NW,500)
          WRITE(NW,500)
    800   RETURN
          END

      $ENTRY
```

EXTREME VALUES OF STRAINS AND MOMENTS DUE TO
UNIFORM D.L.= 400.0 PSF + UNIFORM L.L.= 0.0 PSF
FIRST LEVEL

GAGE NO	STRAINS X-DIR. INX10^{-6}	STRAINS Y-DIR. INX10^{-6}	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.	GAGE NO	STRAINS X-DIR. INX10^{-6}	STRAINS Y-DIR. INX10^{-6}	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.
1	210.0	930.0	11.2	21.0	32	-222.0	-412.0	-7.7	-10.3
2	-34.0	444.0	2.5	9.1	33	-912.0	1776.0	-6.1	30.5
3	-386.0	-1110.0	-16.2	-26.1	34	-1992.0	1012.0	-34.3	6.5
4	-386.0	-878.0	-14.5	-21.2	35	-1558.0	-2168.0	-48.5	-56.8
5	432.0	710.0	14.3	18.0	36	-1558.0	-1404.0	-42.9	-40.8
6	-72.0	1372.0	8.6	28.2	37	-1654.0	1540.0	-23.4	20.1
7	-240.0	1104.0	3.1	21.4	38	-408.0	2434.0	9.3	48.0
8	-612.0	-686.0	-17.9	-18.9	39	-1130.0	2088.0	-8.4	35.5
9	-612.0	404.0	-9.9	4.0	40	-698.0	-906.0	-21.3	-24.1
10	862.0	1058.0	25.8	28.5	41	-698.0	784.0	-8.9	11.3
11	384.0	-1828.0	-5.4	-35.5	42	-670.0	1582.0	-2.4	28.2
12	730.0	1400.0	25.6	34.7	43	-416.0	-3388.0	-33.6	-74.0
13	142.0	-558.0	-1.1	-10.6	44	-338.0	2318.0	9.9	46.1
14	1414.0	818.0	35.6	27.5	45	-338.0	-466.0	-10.5	-12.2
15	2110.0	-952.0	37.2	-4.5	46	-188.0	1128.0	4.3	22.3
16	1458.0	1424.0	41.0	40.5	47	1342.0	-1594.0	16.4	-23.6
17	1154.0	-120.0	23.3	5.9	48	462.0	2088.0	25.0	47.1
18	1418.0	866.0	36.1	28.5	49	586.0	172.0	13.5	7.9
19	1970.0	284.0	43.4	20.4	50	552.0	1058.0	19.3	26.2
20	2210.0	-804.0	40.4	-0.6	51	1164.0	224.0	26.0	13.2
21	1800.0	764.0	43.3	29.2	52	1680.0	-1332.0	25.4	-15.6
22	1222.0	1640.0	37.6	43.3	53	610.0	732.0	18.1	19.8
23	1284.0	1366.0	36.9	38.0	54	534.0	2086.0	26.5	47.6
24	1194.0	94.0	25.7	10.7	55	816.0	1706.0	29.6	41.7
25	1116.0	1174.0	32.0	32.8	56	804.0	226.0	18.5	10.6
26	1676.0	-1256.0	25.9	-14.0	57	-386.0	-994.0	-15.4	-23.7
27	1010.0	1858.0	34.8	46.3	58	-2292.0	-1784.0	-61.1	-54.2
28	858.0	1778.0	31.0	43.5	59	-812.0	-1784.0	-30.1	-43.3
29	-454.0	-142.0	-10.6	-6.3	60	-616.0	-140.0	-13.9	-7.4
30	-96.0	-2950.0	-23.6	-62.5	61	-848.0	-62.0	-18.2	-7.5
31	-164.0	2238.0	13.0	45.7	62	-534.0	-62.0	-11.6	-5.2

TABLE B.1

EXTREME VALUES OF STRAINS AND MOMENTS DUE TO

. UNIFORM D.L.= 400.0 PSF + UNIFORM L.L.= 0.0 PSF

SECOND LEVEL

GAGE NO	STRAINS X-DIR. INX10^{-6}	STRAINS Y-DIR. INX10^{-6}	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.	GAGE NO	STRAINS X-DIR. INX10^{-6}	STRAINS Y-DIR. INX10^{-6}	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.
1	382.0	700.0	13.1	17.5	32	-208.0	-444.0	-7.6	-10.8
2	146.0	218.0	4.7	5.6	33	-910.0	1484.0	-8.2	24.4
3	116.0	-1270.0	-6.9	-25.8	34	-2268.0	670.0	-42.6	-2.6
4	116.0	-1122.0	-5.8	-22.7	35	-1698.0	-1972.0	-50.0	-53.8
5	634.0	876.0	19.7	23.0	36	-1698.0	-2074.0	-50.8	-55.9
6	90.0	1828.0	15.3	39.0	37	-1900.0	1464.0	-29.1	16.7
7	38.0	1664.0	13.0	35.1	38	-724.0	2580.0	3.7	48.7
8	80.0	142.0	2.7	3.6	39	-1244.0	2368.0	-8.7	40.5
9	80.0	96.0	2.4	2.6	40	-988.0	460.0	-17.3	2.4
10	996.0	674.0	25.8	21.4	41	-988.0	194.0	-19.3	-3.2
11	1148.0	-1956.0	9.7	-32.6	42	-778.0	1362.0	-6.3	22.8
12	870.0	1788.0	31.3	43.8	43	-432.0	-3674.0	-36.0	-80.1
13	624.0	-166.0	11.9	1.1	44	-674.0	2528.0	4.4	48.0
14	1840.0	616.0	43.1	26.4	45	-536.0	-146.0	-12.3	-7.0
15	2410.0	-1060.0	42.7	-4.5	46	334.0	912.0	13.7	21.6
16	1556.0	1820.0	45.9	49.5	47	1148.0	-1904.0	10.1	-31.5
17	1392.0	146.0	30.2	13.3	48	236.0	2220.0	21.2	48.2
18	1630.0	668.0	39.0	25.9	49	378.0	312.0	10.2	9.3
19	2110.0	28.0	44.4	16.1	50	434.0	790.0	14.9	19.7
20	2388.0	-1082.0	42.1	-5.2	51	966.0	-24.0	20.1	6.6
21	2020.0	702.0	47.5	29.5	52	1476.0	-1578.0	19.4	-22.2
22	1418.0	1910.0	43.7	50.4	53	638.0	738.0	18.8	20.1
23	1644.0	1804.0	47.7	49.8	54	366.0	2172.0	23.6	48.2
24	1402.0	154.0	30.5	13.5	55	576.0	2024.0	26.9	46.6
25	1152.0	984.0	31.3	29.1	56	594.0	380.0	15.2	12.3
26	1732.0	-1510.0	25.2	-18.9	57	116.0	-1200.0	-6.4	-24.3
27	904.0	2148.0	34.7	51.6	58	-2012.0	-2008.0	-56.9	-56.8
28	966.0	1266.0	29.5	33.6	59	-1396.0	-2008.0	-44.0	-52.3
29	-422.0	132.0	-7.9	-0.3	60	80.0	116.0	2.5	3.0
30	-370.0	-3256.0	-31.6	-70.9	61	-1000.0	328.0	-18.5	-0.5
31	-214.0	2386.0	13.0	48.4	62	-966.0	328.0	-17.8	-0.2

TABLE B.2

STRAINS AND MOMENTS DUE TO

UNIFORM D.L.= 86.2 PSF + UNIFORM L.L.= 313.8 PSF ON MARKED PANELS

PATTERN LOADING NO. 1 FIRST LEVEL

GAGE NO	STRAINS X-DIR. INX10^{-6}	STRAINS Y-DIR. INX10^{-6}	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.	GAGE NO	STRAINS X-DIR. INX10^{-6}	STRAINS Y-DIR. INX10^{-6}	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.
1	294.7	-326.8	3.8	-4.7	32	-248.7	-531.2	-9.1	-13.0
2	173.1	-368.7	0.9	-6.5	33	-524.5	-153.9	-12.1	-7.1
3	-230.7	-328.6	-7.2	-8.6	34	-1055.3	-329.5	-24.5	-14.6
4	-230.7	-628.5	-9.4	-14.9	35	-924.1	-901.8	-26.0	-25.7
5	532.4	1080.3	19.1	26.5	36	-924.1	-1134.1	-27.7	-30.5
6	-203.8	1656.0	7.9	33.2	37	-1089.2	1828.7	-9.4	30.3
7	-335.7	1234.2	2.0	23.4	38	-271.5	2612.9	13.5	52.7
8	-630.8	-773.9	-18.9	-20.8	39	-1104.9	2131.9	-7.5	36.6
9	-630.8	419.7	-10.1	4.2	40	-721.5	-1056.6	-22.9	-27.4
10	509.0	-490.6	7.1	-6.5	41	-721.5	710.3	-9.9	9.6
11	242.8	-1046.6	-2.6	-20.1	42	-368.8	-267.9	-9.7	-8.3
12	584.1	1709.1	24.8	40.1	43	-183.8	-2120.2	-19.4	-45.8
13	123.2	-570.6	-1.6	-11.0	44	-232.9	2534.5	13.7	51.4
14	794.2	-550.2	12.6	-5.7	45	-358.4	-616.6	-12.0	-15.5
15	1372.6	-534.6	24.8	-1.1	46	0.3	-532.0	-3.9	-11.1
16	1221.1	1791.1	38.7	46.5	47	861.9	-999.3	10.7	-14.6
17	1116.3	-101.2	22.6	6.1	48	435.3	2361.0	26.4	52.7
18	647.6	-564.9	9.4	-7.1	49	631.5	46.5	13.6	5.6
19	1130.6	-695.1	18.6	-6.3	50	194.3	-576.9	-0.2	-10.7
20	1391.0	-483.9	25.6	0.1	51	643.1	-844.5	7.3	-13.0
21	1346.6	1272.4	37.5	36.5	52	1030.4	-858.2	15.3	-10.4
22	1069.8	1930.3	36.6	48.3	53	459.4	1331.4	19.4	31.3
23	1139.7	1488.4	34.8	39.5	54	493.2	2241.3	26.8	50.6
24	1201.8	25.0	25.4	9.3	55	806.6	1775.0	29.9	43.1
25	551.2	-424.8	8.4	-4.9	56	873.0	91.1	19.0	8.3
26	1067.2	-750.8	16.9	-7.9	57	-233.8	-477.8	-8.4	-11.7
27	903.3	2149.8	34.7	51.7	58	-1363.2	-1016.8	-36.0	-31.3
28	839.2	-54.6	17.2	5.0	59	-487.2	-1016.8	-17.7	-24.9
29	-229.6	-204.8	-6.3	-6.0	60	-633.3	-177.7	-14.6	-8.4
30	-50.5	-1814.0	-14.4	-38.4	61	-884.1	-175.0	-19.8	-10.1
31	-96.5	2437.3	15.8	50.4	62	-540.3	-175.0	-12.6	-7.6

TABLE B.3

STRAINS AND MOMENTS DUE TO

UNIFORM D.L.= 86.2 PSF + UNIFORM L.L.= 313.8 PSF ON MARKED PANELS

PATTERN LOADING NO. 2 FIRST LEVEL

GAGE NO	STRAINS X-DIR. INX10^{-6}	STRAINS Y-DIR. INX10^{-6}	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.	GAGE NO	STRAINS X-DIR. INX10^{-6}	STRAINS Y-DIR. INX10^{-6}	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.
1	-39.5	1457.2	9.9	30.2	32	-21.2	30.5	-0.2	0.5
2	-214.4	908.4	2.2	17.5	33	-584.1	2312.6	4.7	44.2
3	-238.5	-1020.6	-12.5	-23.1	34	-1366.0	1559.6	-17.2	22.7
4	-238.5	-438.7	-8.2	-10.9	35	-969.6	-1733.4	-33.0	-43.4
5	-7.3	-217.3	-1.7	-4.6	36	-969.6	-572.4	-24.5	-19.1
6	116.3	11.7	2.5	1.1	37	-921.3	43.2	-19.0	-5.9
7	44.0	107.7	1.7	2.6	38	-224.4	345.7	-2.2	5.6
8	-113.1	-60.0	-2.8	-2.1	39	-268.6	406.0	-2.7	6.5
9	-113.1	71.4	-1.8	0.7	40	-126.9	-44.6	-3.0	-1.9
10	538.8	1776.6	24.3	41.2	41	-126.9	242.7	-0.9	4.2
11	224.0	-1175.3	-3.9	-23.0	42	-445.6	2190.8	6.7	42.6
12	303.2	-7.4	6.3	2.1	43	-321.9	-1997.9	-21.4	-44.2
13	49.4	-107.7	0.2	-1.9	44	-178.0	283.0	-1.7	4.6
14	924.5	1544.4	30.7	39.1	45	-52.4	50.2	-0.7	0.7
15	1192.1	-622.5	20.4	-4.3	46	-228.8	1903.1	9.2	38.2
16	551.1	-60.3	11.1	2.8	47	769.3	-938.2	9.2	-14.0
17	286.3	-44.7	5.7	1.2	48	126.2	177.0	3.9	4.6
18	1076.0	1617.6	34.4	41.8	49	80.8	162.6	2.9	4.0
19	1263.9	1040.3	34.1	31.1	50	476.7	1862.9	23.6	42.5
20	1295.3	-493.3	23.5	-0.8	51	771.7	1116.8	24.4	29.1
21	841.3	-343.7	15.1	-1.0	52	1011.6	-760.9	15.6	-8.5
22	415.5	63.2	9.2	4.4	53	282.1	-441.6	2.7	-7.2
23	421.0	172.0	10.1	6.7	54	155.9	294.2	5.4	7.3
24	249.5	89.3	5.9	3.7	55	185.3	298.6	6.1	7.6
25	805.3	1851.8	30.4	44.7	56	104.2	183.6	3.5	4.6
26	969.9	-775.9	14.6	-9.1	57	-235.4	-730.4	-10.3	-17.0
27	324.3	108.6	7.6	4.7	58	-1422.8	-1151.7	-38.3	-34.6
28	203.7	2215.8	20.5	47.9	59	-499.8	-1151.7	-18.9	-27.8
29	-322.2	32.2	-6.5	-1.7	60	-115.5	7.5	-2.4	-0.7
30	-66.2	-1771.7	-14.4	-37.6	61	-146.7	99.6	-2.3	1.0
31	-102.8	283.0	-0.1	5.2	62	-108.8	99.6	-1.5	1.3

TABLE B.4

STRAINS AND MOMENTS DUE TO

UNIFORM D.L.= 86.2 PSF + UNIFORM L.L.= 313.8 PSF ON MARKED PANELS

PATTERN LOADING NO. 3 FIRST LEVEL

GAGE NO	STRAINS X-DIR. $IN \times 10^{-6}$	STRAINS Y-DIR. $IN \times 10^{-6}$	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.	GAGE NO	STRAINS X-DIR. $IN \times 10^{-6}$	STRAINS Y-DIR. $IN \times 10^{-6}$	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.
1	216.3	997.5	11.8	22.5	32	115.3	-284.9	0.3	-5.1
2	-156.4	481.7	0.3	8.9	33	-635.9	1117.0	-5.1	18.7
3	-450.3	-1194.7	-18.2	-28.3	34	-1246.7	670.0	-21.2	4.9
4	-450.3	-934.5	-16.3	-22.9	35	-1048.1	-1207.8	-30.8	-33.0
5	-305.4	669.2	-1.5	11.8	36	-1048.1	-792.1	-27.8	-24.3
6	11.2	1313.9	9.9	27.6	37	-1004.4	1005.0	-13.7	13.7
7	-232.2	1038.1	2.7	20.0	38	-302.9	1550.7	5.0	30.3
8	-608.9	-671.9	-17.7	-18.5	39	-640.5	1377.2	-3.3	24.2
9	-608.9	306.7	-10.5	2.0	40	-395.2	-507.5	-12.0	-13.5
10	874.6	1108.2	26.4	29.6	41	-395.2	602.0	-3.9	9.7
11	376.2	-1897.0	-6.0	-37.0	42	-889.7	919.9	-11.9	12.7
12	748.8	1302.7	25.2	32.8	43	-452.1	-1787.6	-22.6	-40.8
13	148.3	-666.3	-1.8	-12.9	44	-614.1	1375.0	-2.8	24.3
14	1738.8	778.8	42.1	29.1	45	-338.0	-20.4	-7.2	-2.9
15	2221.4	-876.7	40.1	-2.1	46	-875.2	600.8	-13.9	6.2
16	1599.2	1234.2	42.6	37.6	47	-104.6	-847.2	-8.4	-18.5
17	1321.9	-220.4	26.1	5.1	48	-606.5	1174.8	-4.1	20.2
18	1829.1	740.5	43.7	28.9	49	-291.1	292.8	-4.0	4.0
19	2200.6	290.3	48.2	22.2	50	-736.1	551.2	-11.4	6.1
20	2423.4	-609.4	46.3	5.0	51	-405.0	92.2	-7.8	-1.0
21	2036.9	676.1	47.6	29.1	52	-23.9	-706.0	-5.7	-15.0
22	1631.5	1348.2	44.1	40.2	53	-378.5	386.8	-5.1	5.3
23	1600.9	1133.8	41.9	35.5	54	-682.0	1229.3	-5.3	20.8
24	1451.3	-0.1	30.4	10.6	55	-483.1	971.7	-3.0	16.8
25	1556.9	897.9	39.2	30.2	56	-233.1	315.4	-2.6	4.9
26	1974.1	-832.4	35.3	-3.0	57	-448.8	-1066.2	-17.2	-25.6
27	1392.8	1450.1	39.8	40.6	58	-1578.1	-997.9	-40.4	-32.5
28	1200.0	1206.9	34.0	34.1	59	-510.8	-997.9	-18.0	-24.7
29	123.4	-242.4	0.8	-4.2	60	-612.9	-183.9	-14.2	-8.3
30	252.3	-1936.4	-8.9	-38.7	61	-443.2	46.3	-8.9	-2.3
31	259.6	1546.1	16.8	34.3	62	-344.2	46.3	-6.9	-1.6

TABLE B.5

STRAINS AND MOMENTS DUE TO
UNIFORM D.L.= 86.2 PSF + UNIFORM L.L.= 313.8 PSF ON MARKED PANELS
PATTERN LOADING NO. 4. FIRST LEVEL

GAGE NO	STRAINS X-DIR. $IN \times 10^{-6}$	STRAINS Y-DIR. $IN \times 10^{-6}$	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.	GAGE NO	STRAINS X-DIR. $IN \times 10^{-6}$	STRAINS Y-DIR. $IN \times 10^{-6}$	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.
1	39.0	132.9	1.8	3.1	32	-385.2	-215.9	-9.7	-7.3
2	115.1	58.0	2.8	2.1	33	-472.7	1041.7	-2.3	18.4
3	-18.9	-154.5	-1.5	-3.4	34	-1174.6	560.1	-20.5	3.1
4	-18.9	-132.7	-1.4	-2.9	35	-845.7	-1427.4	-28.2	-36.1
5	830.5	193.8	18.8	10.1	36	-845.7	-914.5	-24.4	-25.4
6	-98.7	353.7	0.5	6.7	37	-1006.0	866.9	-14.7	10.8
7	-59.6	303.8	1.0	5.9	38	-193.0	1407.9	6.3	28.1
8	-135.0	-162.0	-4.0	-4.4	39	-733.0	1160.7	-6.8	18.9
9	-135.0	184.3	-1.5	2.9	40	-453.2	-593.8	-13.8	-15.8
10	173.2	177.8	4.9	5.0	41	-453.2	351.0	-6.9	4.0
11	90.6	-324.9	-0.5	-6.1	42	75.3	1003.0	8.9	21.6
12	138.5	399.0	5.8	9.4	43	-53.6	-2330.5	-18.2	-49.2
13	24.3	-12.0	0.4	-0.1	44	203.3	1442.5	14.8	31.7
14	-20.1	215.5	1.2	4.4	45	-72.8	-546.0	-5.5	-12.0
15	343.3	-280.5	5.1	-3.4	46	646.7	770.3	19.2	20.9
16	173.0	496.7	7.3	11.7	47	1735.8	-1090.4	28.4	-10.1
17	80.8	74.6	2.2	2.2	48	1168.0	1363.1	34.5	37.1
18	-105.5	312.1	0.1	5.8	49	1003.4	-83.7	20.4	5.6
19	193.9	54.9	4.5	2.6	50	1407.1	734.8	34.9	25.7
20	262.9	-367.8	2.8	-5.8	51	1819.8	180.1	39.4	17.1
21	151.0	252.5	5.0	6.4	52	2066.0	-913.1	36.6	-4.0
22	-146.2	645.3	1.7	12.4	53	1119.9	502.9	27.1	18.7
23	-40.2	526.6	3.0	10.7	54	1331.1	1306.2	37.5	37.1
24	-0.0	114.4	0.8	2.4	55	1475.0	1101.9	39.0	33.9
25	-200.4	529.1	-0.3	9.6	56	1210.4	-40.7	25.1	8.0
26	63.1	-694.3	-3.8	-14.1	57	-20.4	-142.0	-1.5	-3.1
27	-165.2	808.3	2.5	15.7	58	-1207.8	-1170.5	-33.9	-33.4
28	-157.1	954.3	3.7	18.8	59	-476.2	-1170.5	-18.6	-28.0
29	-675.2	69.8	-13.6	-3.5	60	-135.9	13.8	-2.7	-0.7
30	-369.0	-1649.3	-19.8	-37.3	61	-587.5	-121.6	-13.2	-6.9
31	-459.0	1174.2	-1.0	21.2	62	-304.9	-121.6	-7.3	-4.8

TABLE B.6

STRAINS AND MOMENTS DUE TO
UNIFORM D.L.= 86.2 PSF + UNIFORM L.L.= 313.8 PSF ON MARKED PANELS
PATTERN LOADING NO. 5 FIRST LEVEL

GAGE NO	STRAINS X-DIR. $IN \times 10^{-6}$	STRAINS Y-DIR. $IN \times 10^{-6}$	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.	GAGE NO	STRAINS X-DIR. $IN \times 10^{-6}$	STRAINS Y-DIR. $IN \times 10^{-6}$	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.
1	89.2	1190.5	10.6	25.6	32	-223.6	-393.2	-7.6	-9.9
2	-158.0	509.9	0.4	9.5	33	-766.1	2045.9	-1.0	37.2
3	-403.3	-1263.8	-17.7	-29.4	34	-1962.2	1088.9	-33.1	8.4
4	-403.3	-1011.4	-15.9	-24.1	35	-1594.1	-2378.2	-50.8	-61.5
5	392.8	604.9	12.7	15.6	36	-1594.1	-1548.3	-44.7	-44.1
6	-65.7	1321.8	8.3	27.2	37	-1730.9	1422.3	-25.8	17.1
7	-232.2	1104.0	3.2	21.4	38	-425.3	2374.4	8.5	46.6
8	-608.9	-577.7	-17.0	-16.6	39	-1136.3	2055.1	-8.7	34.7
9	-608.9	377.3	-10.0	3.4	40	-696.4	-844.8	-20.8	-22.8
10	689.4	1403.2	24.7	34.5	41	-696.4	729.1	-9.2	10.2
11	343.2	-1988.0	-7.4	-39.1	42	-564.9	1886.4	2.0	35.4
12	725.3	1341.9	25.0	33.4	43	-442.7	-3651.6	-36.0	-79.7
13	127.9	-531.3	-1.2	-10.2	44	-349.0	2256.8	9.2	44.7
14	1192.8	1164.7	33.5	33.1	45	-355.3	-455.0	-10.8	-12.1
15	2066.1	-1055.6	35.5	-7.0	46	-217.8	1507.7	6.5	30.0
16	1451.7	1370.7	40.5	39.4	47	1385.9	-1776.0	16.0	-27.0
17	1138.3	-109.0	23.0	6.1	48	454.2	2004.8	24.2	45.3
18	1262.7	1225.3	35.4	34.9	49	578.2	172.0	13.4	7.8
19	1831.9	384.4	41.2	21.5	50	536.3	1451.8	21.9	34.3
20	2236.7	-931.1	40.0	-3.1	51	1145.2	326.0	26.4	15.2
21	1785.9	640.0	42.1	26.5	52	1769.4	-1484.2	26.2	-18.1
22	1218.9	1577.2	37.1	42.0	53	592.7	587.7	16.7	16.7
23	1266.7	1353.4	36.5	37.6	54	545.0	2010.7	26.2	46.1
24	1187.7	97.1	25.6	10.7	55	803.4	1687.2	29.2	41.2
25	1018.7	1501.9	32.4	38.9	56	794.6	230.7	18.3	10.7
26	1671.3	-1409.8	24.7	-17.3	57	-400.1	-1138.3	-16.7	-26.8
27	1003.7	1789.0	34.1	44.8	58	-2139.1	-1962.9	-63.4	-58.3
28	850.2	2055.7	32.9	49.3	59	-835.5	-1962.9	-31.9	-47.2
29	-352.0	11.8	-7.3	-2.3	60	-612.9	-100.8	-13.6	-6.6
30	-128.9	-3188.5	-26.1	-67.7	61	-840.2	-57.3	-18.0	-7.4
31	-173.4	2173.7	12.3	44.3	62	-538.7	-57.3	-11.7	-5.2

TABLE B.7

STRAINS AND MOMENTS DUE TO

UNIFORM D.L.= 86.2 PSF + UNIFORM L.L.= 313.8 PSF ON MARKED PANELS

PATTERN LOADING NO. 6 FIRST LEVEL

GAGE NO	STRAINS X-DIR. INX10^{-6}	STRAINS Y-DIR. INX10^{-6}	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.	GAGE NO	STRAINS X-DIR. INX10^{-6}	STRAINS Y-DIR. INX10^{-6}	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.
1	225.7	906.5	11.4	20.6	32	-322.4	-470.1	-10.2	-12.2
2	-30.9	414.2	2.4	8.5	33	-941.8	1807.4	-6.5	31.0
3	-320.1	-1117.8	-14.9	-25.8	34	-2181.8	1035.5	-38.1	5.7
4	-320.1	-892.1	-13.2	-21.0	35	-1649.0	-2262.1	-51.1	-59.5
5	606.2	688.0	17.7	18.9	36	-1649.0	-1458.9	-45.2	-42.7
6	-62.6	1353.2	8.6	27.9	37	-1818.7	1519.6	-27.0	18.5
7	-229.0	1082.0	3.1	21.0	38	-406.4	2390.1	9.0	47.1
8	-563.4	-678.2	-16.8	-18.3	39	-1239.8	2026.8	-11.1	33.4
9	-563.4	380.5	-9.0	3.8	40	-774.9	-946.8	-23.2	-25.5
10	860.4	1039.2	25.6	28.1	41	-774.9	702.4	-11.1	9.0
11	404.4	-1818.6	-4.9	-35.1	42	-657.4	1596.1	-2.1	28.6
12	736.3	1373.3	25.5	34.2	43	-474.1	-3475.9	-35.4	-76.3
13	178.1	-536.0	-0.2	-9.9	44	-345.8	2242.7	9.2	44.4
14	1398.3	805.4	35.2	27.1	45	-381.9	-586.8	-12.3	-15.1
15	2083.3	-966.1	36.6	-5.0	46	48.9	1090.3	9.0	23.2
16	1432.9	1408.3	40.3	40.0	47	1451.8	-1504.6	19.4	-20.9
17	1136.7	-120.0	22.9	5.8	48	625.2	1899.7	27.0	44.4
18	1386.6	862.9	35.4	28.2	49	691.1	29.2	14.7	5.7
19	1899.4	268.3	41.8	19.5	50	970.9	913.7	27.0	26.3
20	2191.2	-868.3	39.5	-2.1	51	1487.2	206.7	32.7	15.2
21	1738.8	727.9	41.8	28.0	52	1873.0	-1113.9	31.1	-9.6
22	1200.0	1627.4	37.1	42.9	53	854.8	663.0	22.8	20.2
23	1235.4	1350.3	35.8	37.3	54	899.6	1648.2	30.9	41.1
24	1157.9	92.4	24.9	10.4	55	1131.4	1411.0	34.0	37.9
25	1036.0	1158.3	30.2	31.9	56	1003.3	81.7	21.6	9.1
26	1605.4	-1318.8	24.0	-15.9	57	-320.1	-1003.4	-14.1	-23.4
27	956.7	1850.2	33.6	45.8	58	-2420.7	-1859.3	-64.3	-56.7
28	785.8	1741.9	29.2	42.3	59	-859.1	-1859.3	-31.6	-45.2
29	-567.0	-165.5	-13.1	-7.6	60	-565.8	-144.7	-12.9	-7.2
30	-207.4	-3074.0	-26.9	-65.9	61	-935.9	-123.2	-20.5	-9.4
31	-228.3	2211.3	11.4	44.7	62	-593.6	-123.2	-13.3	-6.9

TABLE B.8

EXTREME VALUES OF STRAINS AND MOMENTS DUE TO
UNIFORM D.L.= 86.2 PSF + UNIFORM L.L.= 313.8 PSF
FIRST LEVEL

GAGE NO	STRAINS X-DIR. INX10^{-6}	STRAINS Y-DIR. INX10^{-6}	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.	GAGE NO	STRAINS X-DIR. INX10^{-6}	STRAINS Y-DIR. INX10^{-6}	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.
1	216.3	1457.2	11.8	30.2	32	-322.4	-531.2	-10.2	-13.0
2	-156.4	908.4	0.3	17.5	33	-941.8	2312.6	-6.5	44.2
3	-450.3	-1263.8	-18.2	-29.4	34	-2181.8	1559.6	-38.1	22.7
4	-450.3	-1011.4	-16.3	-24.1	35	-1649.0	-2378.2	-51.1	-61.5
5	-305.4	1080.3	-1.5	26.5	36	-1649.0	-1548.3	-45.2	-44.1
6	11.2	1656.0	9.9	33.2	37	-1818.7	1828.7	-27.0	30.3
7	-232.2	1234.2	2.7	23.4	38	-406.4	2612.9	9.0	52.7
8	-608.9	-773.9	-17.7	-20.8	39	-1239.8	2131.9	-11.1	36.6
9	-608.9	419.7	-10.5	4.2	40	-774.9	-1056.6	-23.2	-27.4
10	874.6	1776.6	26.4	41.2	41	-774.9	710.3	-11.1	9.6
11	376.2	-1988.0	-6.0	-39.1	42	-657.4	2190.8	-2.1	42.6
12	748.8	1709.1	25.2	40.1	43	-474.1	-3651.6	-35.4	-79.7
13	148.3	-570.6	-1.8	-11.0	44	-345.8	2534.5	9.2	51.4
14	1738.8	1544.4	42.1	39.1	45	-381.9	-616.6	-12.3	-15.5
15	2221.4	-1055.6	40.1	-7.0	46	646.7	1903.1	19.2	38.2
16	1599.2	1791.1	42.6	46.5	47	1735.8	-1776.0	28.4	-27.0
17	1321.9	-101.2	26.1	6.1	48	1168.0	2361.0	34.5	52.7
18	1829.1	1617.6	43.7	41.8	49	1003.4	46.5	20.4	5.6
19	2200.6	1040.3	48.2	31.1	50	1407.1	1862.9	34.9	42.5
20	2423.4	-931.1	46.3	-3.1	51	1819.8	1116.8	39.4	29.1
21	2036.9	1272.4	47.6	36.5	52	2066.0	-1484.2	36.6	-18.1
22	1631.5	1930.3	44.1	48.3	53	1119.9	1331.4	27.1	31.3
23	1600.9	1488.4	41.9	39.5	54	1331.1	2241.3	37.5	50.6
24	1451.3	25.0	30.4	9.3	55	1475.0	1775.0	39.0	43.1
25	1556.9	1851.8	39.2	44.7	56	1210.4	91.1	25.1	8.3
26	1974.1	-1409.8	35.3	-17.3	57	-448.8	-1138.3	-17.2	-26.8
27	1392.8	2149.8	39.8	51.7	58	-2420.7	-1962.9	-64.3	-58.3
28	1200.0	-54.6	34.0	5.0	59	-859.1	-1962.9	-31.6	-47.2
29	-567.0	32.2	-13.1	-1.7	60	-612.9	-177.7	-14.2	-8.4
30	-207.4	-3188.5	-26.9	-67.7	61	-935.9	-175.0	-20.5	-10.1
31	-228.3	2437.3	11.4	50.4	62	-593.6	-175.0	-13.3	-7.6

TABLE B.9

STRAINS AND MOMENTS DUE TO

UNIFORM D.L.= 86.2 PSF + UNIFORM L.L.= 313.8 PSF ON MARKED PANELS

PATTERN LOADING NO. 1 SECOND LEVEL

GAGE NO	STRAINS X-DIR. INX10^{-6}	STRAINS Y-DIR. INX10^{-6}	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.	GAGE NO	STRAINS X-DIR. INX10^{-6}	STRAINS Y-DIR. INX10^{-6}	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.
1	422.8	-635.2	4.2	-10.2	32	-214.3	-462.8	-7.9	-11.3
2	227.6	-655.9	-0.0	-12.1	33	-511.5	-392.5	-13.6	-12.0
3	122.3	-637.7	-2.1	-12.5	34	-1281.1	-604.0	-31.3	-22.0
4	122.3	-667.0	-2.3	-13.1	35	-960.6	-1195.3	-28.9	-32.1
5	428.5	1364.0	19.0	31.7	36	-960.6	-1496.6	-31.1	-38.4
6	-79.5	2204.6	14.5	45.6	37	-1236.3	1925.3	-11.8	31.3
7	-103.2	1875.8	11.6	38.5	38	-570.2	2914.2	9.4	56.9
8	7.8	149.8	1.3	3.2	39	-1175.0	2559.4	-5.8	45.0
9	7.8	121.1	1.1	2.6	40	-1033.5	475.7	-18.2	2.4
10	575.5	-700.4	6.9	-10.5	41	-1033.5	194.0	-20.2	-3.5
11	785.6	-1212.3	7.6	-19.6	42	-440.7	-469.0	-12.7	-13.1
12	658.2	2195.9	29.9	50.8	43	-201.4	-2360.7	-21.5	-50.9
13	548.7	-151.9	10.4	0.8	44	-529.7	2887.3	10.1	56.6
14	947.2	-819.6	13.8	-10.2	45	-517.2	-175.8	-12.1	-7.5
15	1547.1	-667.8	27.5	-2.6	46	134.7	-782.5	-2.9	-15.4
16	1290.8	2267.2	43.7	57.0	47	724.4	-1226.2	6.2	-20.4
17	1321.4	194.6	29.1	13.8	48	242.3	2665.6	24.6	57.6
18	800.0	-836.7	10.6	-11.7	49	467.4	305.7	12.0	9.8
19	1239.2	-941.6	19.1	-10.6	50	142.2	-841.8	-3.2	-16.6
20	1504.7	-686.6	26.5	-3.4	51	522.0	-1072.1	3.1	-18.6
21	1478.7	1375.1	41.1	39.6	52	879.8	-1011.6	11.0	-14.7
22	1203.0	2360.3	42.5	58.3	53	510.9	1488.0	21.6	34.9
23	1452.6	2045.6	45.4	53.5	54	358.2	2622.3	26.7	57.6
24	1373.8	221.5	30.4	14.7	55	601.1	2243.7	29.0	51.4
25	569.9	-548.9	7.9	-7.3	56	680.3	384.7	17.1	13.0
26	1127.9	-981.2	16.4	-12.3	57	122.3	-655.6	-2.2	-12.8
27	795.7	2577.9	35.6	59.8	58	-1164.7	-1330.2	-34.2	-36.4
28	940.9	-521.1	15.9	-4.0	59	-765.3	-1330.2	-25.8	-33.5
29	-288.6	179.1	-4.7	1.6	60	7.8	131.7	1.1	2.8
30	-192.7	-2074.5	-19.2	-44.9	61	-1036.1	335.8	-19.2	-0.6
31	-79.1	2756.3	18.6	57.2	62	-1022.5	335.8	-19.0	-0.5

TABLE B.10

STRAINS AND MOMENTS DUE TO

UNIFORM D.L.= 86.2 PSF + UNIFORM L.L.= 313.8 PSF ON MARKED PANELS

PATTERN LOADING NO. 2 SECOND LEVEL

GAGE NO	STRAINS X-DIR. $INX10^{-6}$	STRAINS Y-DIR. $INX10^{-6}$	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.	GAGE NO	STRAINS X-DIR. $INX10^{-6}$	STRAINS Y-DIR. $INX10^{-6}$	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.
1	41.5	1486.1	11.8	31.4	32	-38.5	-76.9	-1.4	-1.9
2	-50.1	920.9	5.7	18.9	33	-594.6	2196.3	3.6	41.7
3	18.7	-906.0	-6.3	-18.8	34	-1475.7	1418.4	-20.5	18.9
4	18.7	-696.8	-4.7	-14.5	35	-1103.3	-1201.6	-31.9	-33.3
5	342.2	-299.2	5.0	-3.8	36	-1103.3	-1024.3	-30.6	-29.5
6	188.8	17.4	4.1	1.7	37	-1073.1	-145.8	-23.6	-10.9
7	149.4	146.8	4.2	4.2	38	-309.8	221.8	-4.9	2.4
8	89.4	22.8	2.0	1.1	39	-337.1	318.9	-4.7	4.2
9	89.4	-4.4	1.8	0.6	40	-167.4	83.4	-2.9	0.5
10	635.1	1519.7	24.4	36.5	41	-167.4	41.8	-3.2	-0.4
11	609.8	-1165.2	4.2	-19.9	42	-505.0	2124.5	5.0	40.8
12	399.3	-22.6	8.2	2.5	43	-323.7	-2105.0	-22.2	-46.5
13	209.8	-49.9	4.0	0.5	44	-289.6	185.5	-4.7	1.8
14	1289.3	1568.4	38.5	42.3	45	-134.3	-1.7	-2.8	-1.0
15	1382.3	-620.7	24.4	-2.9	46	271.2	1891.1	19.5	41.6
16	600.5	-55.0	12.2	3.3	47	671.0	-1088.1	6.1	-17.9
17	370.6	-17.2	7.6	2.4	48	44.6	32.8	1.2	1.0
18	1181.3	1648.6	36.8	43.2	49	-8.0	73.5	0.4	1.5
19	1325.5	975.7	34.9	30.2	50	385.4	1802.0	21.3	40.6
20	1398.0	-628.6	24.7	-2.9	51	652.2	1042.9	21.3	26.6
21	976.6	-521.8	16.6	-3.8	52	914.3	-906.5	12.5	-12.3
22	520.5	-38.7	10.6	3.0	53	264.6	-590.9	1.2	-10.4
23	545.7	147.1	12.5	7.1	54	86.7	17.8	1.9	1.0
24	330.4	-34.3	6.7	1.7	55	99.0	216.5	3.7	5.3
25	830.4	1745.0	30.2	42.6	56	41.7	77.2	1.4	1.9
26	977.3	-854.2	14.2	-10.7	57	18.7	-803.0	-5.5	-16.7
27	303.1	33.0	6.6	2.9	58	-1280.8	-1110.5	-35.0	-32.7
28	233.3	2059.9	20.0	44.9	59	-931.6	-1110.5	-27.7	-30.1
29	-224.3	-18.6	-4.8	-2.0	60	89.4	9.3	1.9	0.9
30	-257.0	-1883.1	-19.2	-41.3	61	-179.4	62.8	-3.3	0.0
31	-181.1	143.9	-2.7	1.7	62	-151.7	62.8	-2.7	0.2

TABLE B.11

STRAINS AND MOMENTS DUE TO

UNIFORM D.L.= 86.2 PSF + UNIFORM L.L.= 313.9 PSF ON MARKED PANELS

PATTERN LOADING NO. 3 SECOND LEVEL

GAGE NO	STRAINS X-DIR. IN $\times 10^{-6}$	STRAINS Y-DIR. IN $\times 10^{-6}$	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.	GAGE NO	STRAINS X-DIR. IN $\times 10^{-6}$	STRAINS Y-DIR. IN $\times 10^{-6}$	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.
1	383.6	808.3	14.0	19.7	32	192.1	-274.5	2.0	-4.3
2	131.9	288.6	4.9	7.0	33	-646.4	911.3	-6.9	14.4
3	92.5	-1317.1	-7.7	-26.9	34	-1458.4	406.4	-27.6	-2.2
4	92.5	-1136.1	-6.4	-23.1	35	-1123.7	-1154.6	-32.0	-32.4
5	-454.9	839.9	-3.4	14.3	36	-1123.7	-1184.4	-32.2	-33.1
6	151.2	1729.2	15.8	37.3	37	-1222.2	966.6	-18.5	11.3
7	72.5	1529.1	12.7	32.6	38	-560.8	1676.3	0.5	31.0
8	106.7	25.9	2.4	1.3	39	-781.1	1534.9	-5.1	26.4
9	106.7	9.7	2.3	1.0	40	-592.6	342.3	-9.9	2.8
10	1057.2	752.4	27.7	23.5	41	-592.6	150.1	-11.3	-1.2
11	1209.2	-1973.3	10.9	-32.5	42	-1036.9	792.5	-15.9	9.0
12	984.5	1656.2	32.8	41.9	43	-585.8	-2061.1	-27.4	-47.5
13	660.1	-326.0	11.4	-2.0	44	-826.2	1534.8	-6.1	26.1
14	2048.7	617.6	47.4	28.0	45	-568.9	48.6	-11.6	-3.2
15	2596.7	-936.0	47.5	-0.6	46	-783.1	464.8	-13.0	4.0
16	1797.6	1570.5	49.2	46.1	47	-265.7	-1085.0	-13.5	-24.7
17	1575.6	-7.8	33.0	11.4	48	-744.6	1280.2	-6.2	21.4
18	2023.8	586.4	46.7	27.1	49	-474.0	349.7	-7.4	3.8
19	2489.7	112.7	53.0	20.6	50	-852.6	382.1	-15.1	1.8
20	2595.1	-796.4	48.5	2.3	51	-615.6	-75.8	-13.5	-6.1
21	2393.4	634.5	54.8	30.8	52	-162.0	-903.3	-10.0	-20.1
22	1800.8	1575.8	49.3	46.2	53	-276.7	392.8	-2.9	6.2
23	2039.4	1454.1	53.4	45.4	54	-817.0	1225.9	-8.1	19.7
24	1656.2	12.8	34.8	12.4	55	-710.6	1161.0	-6.4	19.1
25	1649.4	827.1	40.6	29.4	56	-392.9	403.5	-5.3	5.6
26	2086.6	-1000.1	36.4	-5.7	57	92.5	-1229.8	-7.1	-25.1
27	1402.9	1631.8	41.4	44.5	58	-1440.9	-1154.5	-38.7	-34.7
28	1328.4	843.9	34.0	27.4	59	-813.9	-1154.5	-25.5	-30.2
29	221.3	25.3	4.8	2.2	60	106.7	14.0	2.3	1.1
30	138.4	-2087.1	-12.4	-42.7	61	-632.9	246.4	-11.5	0.5
31	295.9	1628.2	18.1	36.3	62	-549.6	246.4	-9.7	1.1

TABLE B.12

STRAINS AND MOMENTS DUE TO
UNIFORM D.L.= 86.2 PSF + UNIFORM L.L.= 313.8 PSF ON MARKED PANELS
PATTERN LOADING NO. 4 SECOND LEVEL

GAGE NO	STRAINS X-DIR. $IN \times 10^{-6}$	STRAINS Y-DIR. $IN \times 10^{-6}$	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.	GAGE NO	STRAINS X-DIR. $IN \times 10^{-6}$	STRAINS Y-DIR. $IN \times 10^{-6}$	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.
1	80.8	42.6	2.0	1.5	32	-444.9	-265.1	-11.3	-8.8
2	45.6	-23.6	0.8	-0.2	33	-459.7	892.5	-3.1	15.3
3	48.5	-226.6	-0.6	-4.4	34	-1298.4	408.0	-24.2	-1.0
4	48.5	-227.7	-0.7	-4.4	35	-940.2	-1242.4	-28.8	-32.9
5	1225.5	224.9	27.3	13.7	36	-940.2	-1336.6	-29.5	-34.9
6	-41.8	492.8	2.7	10.0	37	-1087.3	812.9	-16.8	9.1
7	-26.3	493.5	3.1	10.1	38	-319.2	1459.7	4.0	28.2
8	-9.4	146.7	0.9	3.0	39	-730.9	1343.4	-5.5	22.8
9	-9.4	107.0	0.6	2.2	40	-608.3	216.8	-11.2	0.1
10	153.4	66.8	3.7	2.5	41	-608.3	85.7	-12.1	-2.7
11	186.2	-404.3	0.9	-7.1	42	91.2	863.1	8.2	18.7
12	72.9	517.1	5.3	11.4	43	60.7	-2404.7	-16.4	-49.9
13	98.4	124.3	3.0	3.3	44	6.9	1538.0	11.4	32.3
14	187.8	131.2	4.9	4.1	45	-82.6	-226.0	-3.4	-5.3
15	332.6	-352.4	4.4	-4.9	46	1189.1	643.7	29.6	22.2
16	93.7	641.7	6.7	14.1	47	1661.1	-1229.3	25.8	-13.6
17	116.4	185.2	3.8	4.7	48	1031.5	1418.2	32.0	37.3
18	-42.6	225.5	0.8	4.4	49	933.4	29.6	19.8	7.5
19	75.0	-78.7	1.0	-1.1	50	1380.1	578.2	33.2	22.2
20	307.5	-518.7	2.6	-8.6	51	1789.7	46.6	37.8	14.1
21	61.9	218.7	2.9	5.0	52	1956.1	-1014.7	33.5	-6.9
22	-77.3	745.8	3.8	15.1	53	1052.2	504.2	25.7	18.3
23	-41.1	738.6	4.6	15.2	54	1261.9	1414.2	36.8	38.9
24	48.0	174.4	2.3	4.0	55	1410.7	1299.1	39.1	37.6
25	-249.1	369.0	-2.5	5.9	56	1114.9	58.4	23.8	9.4
26	18.7	-835.3	-5.7	-17.4	57	48.5	-228.8	-0.7	-4.4
27	-304.1	979.1	0.8	18.3	58	-1004.7	-1286.3	-30.5	-34.3
28	-154.3	694.9	1.9	13.4	59	-882.9	-1286.3	-27.9	-33.4
29	-734.2	135.1	-14.4	-2.6	60	-9.4	127.0	0.7	2.6
30	-588.1	-1870.6	-26.0	-43.5	61	-582.6	152.3	-11.1	-1.1
31	-556.0	1272.0	-2.3	22.6	62	-625.5	152.3	-12.0	-1.4

TABLE B.13

STRAINS AND MOMENTS DUE TO
UNIFORM D.L.= 86.2 PSF + UNIFORM L.L.= 313.8 PSF ON MARKED PANELS
PATTERN LOADING NO. 5 SECOND LEVEL

GAGE NO	STRAINS X-DIR. $INX10^{-6}$	STRAINS Y-DIR. $INX10^{-6}$	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.	GAGE NO	STRAINS X-DIR. $INX10^{-6}$	STRAINS Y-DIR. $INX10^{-6}$	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.
1	217.3	1079.7	12.5	24.2	32	-211.1	-461.3	-7.8	-11.2
2	4.8	326.3	2.5	6.9	33	-768.8	1822.9	-2.7	32.6
3	83.1	-1483.4	-9.1	-30.5	34	-2209.9	800.2	-40.4	0.6
4	83.1	-1255.4	-7.5	-25.7	35	-1760.8	-2091.2	-52.2	-56.7
5	641.8	741.1	18.9	20.2	36	-1760.8	-2191.7	-53.0	-58.8
6	99.4	1740.1	14.8	37.2	37	-1959.6	1349.5	-31.2	13.9
7	34.9	1616.9	12.6	34.1	38	-736.6	2500.0	2.9	47.0
8	95.7	142.0	3.0	3.7	39	-1247.1	2317.8	-9.1	39.4
9	95.7	94.4	2.7	2.7	40	-984.9	436.5	-17.4	1.9
10	837.5	1072.5	25.4	28.6	41	-984.9	183.0	-19.3	-3.4
11	1102.5	-2142.7	7.4	-36.8	42	-638.4	1738.6	-0.6	31.7
12	855.9	1701.7	30.4	41.9	43	-432.0	-3907.8	-37.7	-85.0
13	624.0	-161.3	11.9	1.2	44	-689.7	2459.0	3.6	46.5
14	1568.6	1063.2	40.7	33.8	45	-540.7	-135.0	-12.3	-6.8
15	2378.6	-1163.6	41.3	-6.9	46	354.4	1373.3	17.5	31.4
16	1541.9	1735.3	45.0	47.7	47	1193.5	-2045.2	10.0	-34.1
17	1385.7	139.7	30.1	13.1	48	232.9	2129.0	20.5	46.3
18	1435.4	1127.7	38.3	34.1	49	370.2	288.5	9.9	8.8
19	1948.4	164.5	42.0	17.7	50	399.5	1265.4	17.6	29.4
20	2414.7	-1191.8	41.8	-7.3	51	972.3	112.5	21.2	9.5
21	1996.5	584.3	46.1	26.9	52	1570.1	-1711.4	20.3	-24.3
22	1422.7	1822.1	43.2	48.6	53	631.7	576.4	17.5	16.7
23	1622.0	1744.4	46.8	48.4	54	375.4	2073.2	23.1	46.2
24	1398.9	132.0	30.3	13.0	55	576.0	1965.9	26.5	45.4
25	979.4	1247.6	29.7	33.3	56	576.7	347.1	14.6	11.5
26	1733.6	-1634.0	24.3	-21.5	57	83.1	-1372.6	-8.3	-28.1
27	891.4	2052.3	33.7	49.5	58	-2071.6	-2127.2	-59.0	-59.8
28	961.3	1631.6	32.1	41.2	59	-1463.5	-2127.2	-46.3	-55.3
29	-324.7	106.9	-6.0	-0.1	60	95.7	116.0	2.9	3.1
30	-368.4	-3486.6	-33.3	-75.7	61	-987.4	310.7	-18.4	-0.7
31	-232.8	2313.8	12.1	46.8	62	-975.4	310.7	-18.2	-0.6

TABLE B.14

STRAINS AND MOMENTS DUE TO

UNIFORM D.L.= 86.2 PSF + UNIFORM L.L.= 313.8 PSF ON MARKED PANELS

PATTERN LOADING NO. 6 SECOND LEVEL

GAGE NO	STRAINS X-DIR. INX10^{-6}	STRAINS Y-DIR. INX10^{-6}	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.	GAGE NO	STRAINS X-DIR. INX10^{-6}	STRAINS Y-DIR. INX10^{-6}	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.
1	389.8	684.3	13.2	17.2	32	-292.7	-541.3	-10.1	-13.5
2	135.0	196.0	4.3	5.1	33	-916.3	1560.9	-7.8	26.0
3	130.1	-1285.7	-6.7	-26.0	34	-2423.3	732.8	-45.4	-2.4
4	130.1	-1136.1	-5.6	-22.8	35	-1784.3	-2000.2	-52.0	-55.0
5	849.0	850.9	24.0	24.1	36	-1784.3	-2111.7	-52.9	-57.3
6	96.3	1799.8	15.2	38.4	37	-2045.9	1457.7	-32.2	15.5
7	34.9	1642.0	12.8	34.7	38	-698.9	2539.2	4.0	48.1
8	80.0	140.4	2.7	3.5	39	-1355.4	2292.7	-11.6	38.1
9	80.0	97.6	2.4	2.6	40	-1091.6	384.7	-20.0	0.1
10	986.6	661.4	25.5	21.1	41	-1091.6	172.0	-21.6	-4.4
11	1144.9	-1962.3	9.6	-32.7	42	-746.6	1409.1	-5.3	24.0
12	866.9	1758.2	31.1	43.2	43	-458.7	-3733.6	-37.0	-81.6
13	625.6	-162.9	11.9	1.2	44	-685.0	2444.8	3.6	46.2
14	1811.8	620.7	42.5	26.3	45	-581.5	-293.5	-14.3	-10.4
15	2386.5	-1075.7	42.1	-5.0	46	553.7	897.9	18.2	22.9
16	1538.7	1799.6	45.4	49.0	47	1314.3	-1778.5	14.5	-27.6
17	1362.2	139.7	29.6	12.9	48	411.7	1994.1	23.2	44.8
18	1597.1	690.0	38.5	26.2	49	534.9	155.1	12.3	7.2
19	2033.1	10.7	42.7	15.1	50	911.0	667.6	24.0	20.7
20	2373.9	-1122.8	41.5	-6.1	51	1359.8	-20.9	28.3	9.5
21	1941.6	673.8	45.6	28.4	52	1709.8	-1330.1	26.1	-15.3
22	1399.2	1875.5	43.1	49.5	53	790.2	636.0	21.2	19.1
23	1567.1	1756.9	45.7	48.3	54	800.6	1815.8	30.1	43.9
24	1350.2	130.5	29.2	12.6	55	968.3	1661.6	32.5	41.9
25	1064.1	996.6	29.6	28.7	56	859.2	204.3	19.5	10.6
26	1666.1	-1552.4	23.5	-20.3	57	130.1	-1212.6	-6.2	-24.4
27	838.1	2115.1	33.1	50.5	58	-2106.1	-2053.5	-59.2	-58.5
28	895.4	1328.8	28.5	34.4	59	-1472.9	-2053.5	-45.9	-53.8
29	-531.8	91.2	-10.5	-2.0	60	80.0	116.0	2.5	3.0
30	-467.3	-3361.1	-34.4	-73.8	61	-1081.6	279.4	-20.6	-2.1
31	-289.3	2371.9	11.3	47.6	62	-1091.5	279.4	-20.8	-2.2

TABLE B.15

EXTREME VALUES OF STRAINS AND MOMENTS DUE TO
UNIFORM D.L.= 86.2 PSF. + UNIFORM L.L.= 313.8 PSF
SECOND LEVEL

GAGE NO	STRAINS X-DIR. $INX10^{-6}$	STRAINS Y-DIR. $INX10^{-6}$	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.	GAGE NO	STRAINS X-DIR. $INX10^{-6}$	STRAINS Y-DIR. $INX10^{-6}$	MOMENTS X-DIR. LB. IN.	MOMENTS Y-DIR. LB. IN.
1	383.6	1486.1	14.0	31.4	32	-292.7	-462.8	-10.1	-11.3
2	131.9	920.9	4.9	18.9	33	-916.3	2196.3	-7.8	41.7
3	92.5	-1483.4	-7.7	-30.5	34	-2423.3	1418.4	-45.4	18.9
4	92.5	-1255.4	-6.4	-25.7	35	-1784.3	-2091.2	-52.0	-56.7
5	-454.9	1364.0	-3.4	31.7	36	-1784.3	-2191.7	-52.9	-58.8
6	151.2	2204.6	15.8	45.6	37	-2045.9	1925.3	-32.2	31.3
7	72.5	1875.8	12.7	38.5	38	-698.9	2914.2	4.0	56.9
8	106.7	149.8	2.4	3.2	39	-1355.4	2559.4	-11.6	45.0
9	106.7	121.1	2.3	2.6	40	-1091.6	475.7	-20.0	2.4
10	1057.2	1519.7	27.7	36.5	41	-1091.6	194.0	-21.6	-3.5
11	1209.2	-2142.7	10.9	-36.8	42	-746.6	2124.5	-5.3	40.8
12	984.5	2195.9	32.8	50.8	43	-458.7	-3907.8	-37.0	-85.0
13	660.1	-151.9	11.4	0.8	44	-685.0	2887.3	3.6	56.6
14	2048.7	1568.4	47.4	42.3	45	-581.5	-175.8	-14.3	-7.5
15	2596.7	-1163.6	47.5	-6.9	46	1189.1	1991.1	29.6	41.6
16	1797.6	2267.2	49.2	57.0	47	1661.1	-2045.2	25.8	-34.1
17	1575.6	194.6	33.0	13.8	48	1031.5	2665.6	32.0	57.6
18	2023.8	1648.6	46.7	43.2	49	933.4	305.7	19.8	9.8
19	2489.7	975.7	53.0	30.2	50	1380.1	1802.0	33.2	40.6
20	2595.1	-1191.8	48.5	-7.3	51	1789.7	1042.9	37.8	26.6
21	2393.4	1375.1	54.8	39.6	52	1956.1	-1711.4	33.5	-24.3
22	1800.8	2360.3	49.3	58.3	53	1052.2	1488.0	25.7	34.9
23	2039.4	2045.6	53.4	53.5	54	1261.9	2622.3	36.8	57.6
24	1656.2	221.5	34.8	14.7	55	1410.7	2243.7	39.1	51.4
25	1649.4	1745.0	40.6	42.6	56	1114.9	384.7	23.8	13.0
26	2086.6	-1634.0	36.4	-21.5	57	92.5	-1372.6	-7.1	-28.1
27	1402.9	2577.9	41.4	59.8	58	-2106.1	-2127.2	-59.2	-59.8
28	1328.4	-521.1	34.0	-4.0	59	-1472.9	-2127.2	-45.9	-55.3
29	-531.8	-18.6	-10.5	-2.0	60	106.7	131.7	2.3	2.8
30	-467.3	-3486.6	-34.4	-75.7	61	-1081.6	335.8	-20.6	-0.6
31	-289.3	2756.3	11.3	57.2	62	-1091.5	335.8	-20.8	-0.5

TABLE B.16

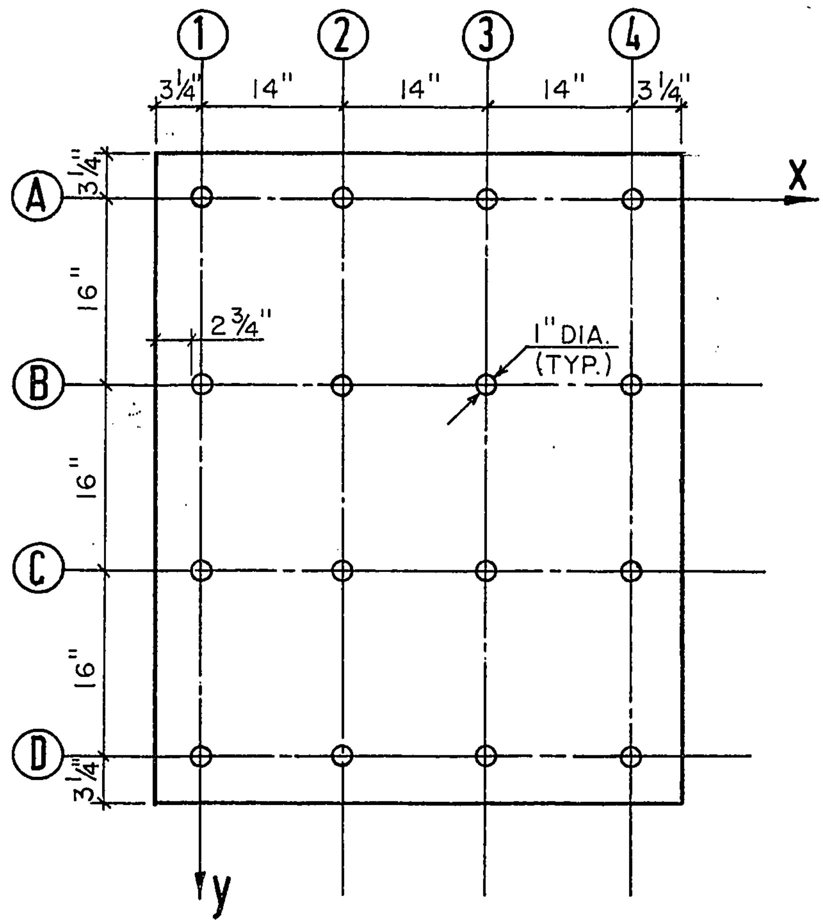
APPENDIX C

EXAMPLE

EXAMPLE "1-A"

Theoretical Design Moments

by ACI 318-71 DIRECT DESIGN METHOD



Design data:

L.L. = 184.6 psf

D.L. = 61.54 psf

- No column capitals or edge beams.
- Storey height, center to center = 10-1/2 in.
- Slab thickness = 0.486 in.
- Direct design method limitations are satisfied.

- Design loads: $w_d = 1.4 \times 61.54 = 86.2$ psf

  $w_1 = 1.7 \times 184.6 = \underline{313.8}$ psf

  $w =$ 400 psf = 2.778 psi

- $I_c = \frac{\pi d^4}{64} = 0.0491 \text{ in}^4$

- $\beta_a = \frac{61.54}{184.60} = 0.33$

- $x = 0.486$ in

- $y = 0.89$ in

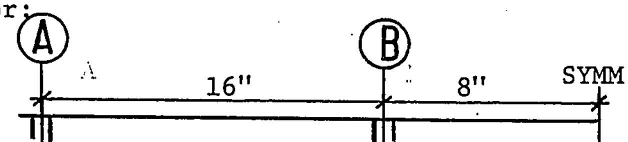
- $c = (1-0.63 \frac{x}{y}) \frac{x^3 y}{3} = 0.02234$

- $\alpha_1 = 0.00$

FIRST LEVEL

- Transverse strips, y-direction

(a) Column lines 2, 3 - Interior:



$I_s = 0.1339$					
$\ell_1, \ell_2, c, \ell_n$	16",14",0.89",15.11"			16,14,0.89,15.11	
ℓ_2/ℓ_1	0.875			0.875	
$M_o = \dfrac{w\ell_2\ell_n^2}{8}$	1109.9 lb.in			1109.9 lb.in	
$K_c(\text{above}) = 4I_c/\ell_c^*$	0.0187			0.0187	
$K_c(\text{below}) = 4I_c/\ell_c^*$	0.0187			0.0187	
ΣK_c	0.0374			0.0374	
$K_t = \Sigma \dfrac{9c}{\ell_2(1 - \frac{c_2}{\ell_2})^3}$ *	0.03498			0.03498	
K_{ec} (ACI eq. 13.5)	0.0181			0.0181	
$K_s = \dfrac{4I_s}{\ell_1}$ *	0.03348		0.03348	0.03348	
$\alpha_{ec} = K_{ec}/\Sigma K_s$	0.5406			0.2703	
$1 + \dfrac{1}{\alpha_{ec}}$	2.85			4.7	
I) % M_o (ACI Section 13.3.3) (for $\alpha_a > 2$)	22.8	53.2	71.5	65	35
Design M slab	-253	+590	-793	-721	+389
M for column strip	-253	+354	-595	-541	+233
M for middle strip	0.00	+236	-198	-180	+156
M for column strip/in	-36.1	+50.6	-85	-77.3	+33.3
M for middle strip/in	0.00	+33.7	-28.3	-25.7	+22.3

*As E is constant it may be omitted.

II) Provisions for effects of pattern loadings (ACI 13.3.6)

	A	16"		B	8"	SYMM
$\alpha_{min} = 2.1$ (ACI Table 13.3.6.1)						
$\alpha_c = \dfrac{\Sigma K_c}{\Sigma K_s}$	1.12			0.56		
$\delta_s = 1 + \dfrac{2-\beta_a}{4+\beta_a}\left(1-\dfrac{\alpha_c}{\alpha_{min}}\right)$	1.18			1.28		
		$\dfrac{1.18+1.28}{2} = 1.23$			1.28	
M for column strip	-172	+435	-514	-476	+298	
M for middle strip	0.00	+290	-90	-136	+200	
M for column strip/in	-24.6	+62.1	-73.4	-68	+42.6	
M for middle strip/in	0.00	+41.4	-12.9	-19.4	+28.6	

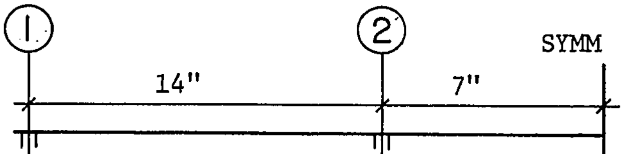
(b) Column lines 1, 4 - Exterior

	(A)	16"		(B)	8" SYMM
$I_s = 0.0981$					
$M_o = \dfrac{w(\ell_2/2)ln^2}{8}$		555 lb.in			555 lb.in
K_c (above)	0.0187			0.0187	
K_c (below)	0.0187			0.0187	
ΣK_c	0.0374			0.0374	
K_t (1/2 for column line "2")	0.0175			0.0175	
K_{ec}	0.012			0.012	
K_s	0.0245		0.0245	0.0245	
$\alpha_{ec} = K_{ec}/\Sigma K_s$	0.49			0.245	
$1 + \dfrac{1}{\alpha_{ec}}$	3.04			5.08	
I) % M_o (for $\beta_a > 2$)	21.4	53.8	71.7	65	35
Design M slab	-119	+299	-398	-361	+194
M for column strip	-119	+179	-299	-271	+116
M for middle strip	0.00	+120	- 99	- 90	+78
II) Provisions for effects of pattern loadings $\alpha_{min} = 2.1$					
$\alpha_c = \dfrac{\Sigma K_c}{\Sigma K_s}$	1.53			0.76	
$\delta_s = 1 + \dfrac{2-\beta_a}{4+\beta_a}\left(1 - \dfrac{\alpha_c}{\alpha_{min}}\right)$	1.10			1.25	
		$\dfrac{1.1+1.25}{2} = 1.175$			
M for column strip		+210			+145
M for middle strip		+141			+98

- Longitudinal strips, x-direction

(a) Column lines B, C - Interior:

	① 14"			② 7"	SYMM
$I_s = 0.153$					
ℓ_1, ℓ_2, c, ℓn	14",16",0.89",13.11"			14,16,0.89,13.11	
ℓ_2/ℓ_1	1.143			1.143	
$M_o = \dfrac{w\ell_2 \ell n^2}{8}$		954.9 lb.in		954.9	
K_c(above) = K_c(below)	0.0187			0.0187	
ΣK_c	0.0374			0.0374	
$K_t = \Sigma \dfrac{9c}{\ell_2(1 - \frac{c_2}{\ell_2})^3}$	0.0298			0.0298	
K_{ec}	0.01659			0.01659	
$K_s = 4I_s/\ell_1$	0.0437		0.0437	0.0437	
$\alpha_{ec} = K_{ec}/\Sigma K_s$	0.3796			0.1898	
$1 + \dfrac{1}{\alpha_{ec}}$	3.63			6.27	
I) % M_o (for $\beta_a > 2$)	17.9	55.3	72.2	65	35
Design M slab	-171	+528	-689	-621	+334
M for column strip	-171	+317	-517	-466	+200
M for middle strip	0.00	+211	-172	-155	+134
M for column strip/in	-24.4	+45.3	-73.9	-66.6	+28.6
M for middle strip/in	0.00	+23.4	-19.1	-17.2	+14.9



II) Provisions for effects of pattern loading (ACI 13.3.6)

$\alpha_{min} = 2.59$

	①			②	SYMM
$\alpha_c = \Sigma K_c / \Sigma K_s$	0.86			0.43	
$\delta_s = 1 + \frac{2-\beta_a}{4+\beta_a}(1 - \frac{\alpha_c}{\alpha_{min}})$	1.26			1.32	
		$\frac{1.26+1.32}{2} = 1.29$			
M for column strip	-79	+409	-425	-402	+264
M for middle strip	0.00	+272	-50	-112	+177
M for column strip/in	-11.3	+58.4	-60.7	-57.4	+37.7
M for middle strip/in	0.00	+30.2	-5.6	-12.4	+19.7

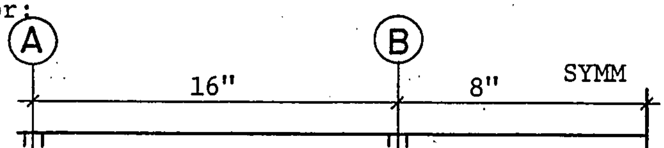
(b) Column lines A, D - Exterior

	①	14"		②	7" SYMM
$I_s = 0.1077$					
M_o		477.5			477.5
K_c(above) = K_c(below)	0.0187			0.0187	
ΣK_c	0.0374			0.0374	
K_t(1/2 for column line "B")	0.0149			0.0149	
K_{ec}	0.0107			0.0107	
K_s	0.03077		0.03077	0.03077	
α_{ec}	0.348			0.174	
$1 + \frac{1}{\alpha_{ec}}$	3.9			5.7	
I) % M_o (for $\beta_a > 2$)	16.7	55.8	72.4	65	35
Design M slab	-80	+266	-346	-310	+167
M for column strip	-80	+160	-260	-233	+100
M for middle strip	0.00	+106	-86	-77	+67
II) Provisions for effects of pattern loadings α_{min} =	2.59				
$\alpha_c = \frac{\Sigma K_c}{\Sigma K_s}$	1.22			0.608	
δ_s	1.20			1.29	
		$\frac{1.20+1.29}{2} = 1.245$			
M for column strip		+199			+129
M for middle strip		+132			+86

SECOND LEVEL

- Transverse strips, y-direction

(a) Column lines 2, 3 - Interior:



$I_s = 0.1339$					
ℓ_1, ℓ_2, c, ℓn	16",14",0.89",15.11"			16",14",0.89",15.11"	
ℓ_2/ℓ_1	0.875			0.875	
$M_o = \dfrac{w\ell_2\ell n^2}{8}$	1101.9 lb.in			1109.9 lb.in	
K_c(above)	0.0			0.0	
K_c(below) $= 4I_c/\ell_c^*$	0.0187			0.0187	
ΣK_c	0.0187			0.0187	
$K_t = \Sigma \dfrac{9c}{\ell_2(1 - \frac{c_2}{\ell_2})^3}$ *	0.03498			0.03498	
K_{ec}(ACI eq. 13-5)	0.01219			0.01219	
$K_s = \dfrac{4I_s}{\ell_1}$ *	0.03348		0.03348	0.03348	
$\alpha_{ec} = K_{ec}/\Sigma K_s$	0.3641			0.182	
$1 + \dfrac{1}{\alpha_{ec}}$	3.75			6.5	
I) % M_o (ACI Section 13.3.3) (for $\beta_a > 2$)	17.3	55.5	72.3	65	35
Design M slab	-192	+616	-802.5	-721	+389
M for column strip	-192	+370	-602	-541	+233
M for middle strip	0.00	+246	-200.5	-180	+156
M for column strip/in	-27.4	+52.9	-86	-77.3	+33.3
M for middle strip/in	0.00	+35.1	-28.6	-25.7	+22.3

*As E is constant it may be omitted.

II) Provisions for effects of pattern loadings (ACI 13.3.6)

	(A)	16"		(B)	8" SYMM
$\alpha_{min} = 2.1$					
$\alpha_c = \dfrac{\Sigma K_c}{\Sigma K_s}$	0.56			0.28	
$\delta_s = 1 + \dfrac{2-\beta_a}{4+\beta_a}\left(1 - \dfrac{\alpha_c}{\alpha_{min}}\right)$	1.28			1.33	
		$\dfrac{1.28+1.33}{2} = 1.305$			
M for column strip	-79	+483	-489	-464	+310
M for middle strip	0.00	+321	-51	-128	+208
M for column strip/in	-11.3	+69	-69.9	-66.3	+44.3
M for middle strip/in	0.00	+45.9	-7.3	-18.3	+29.7

(b) Column lines 1, 4 - Exterior:

	A	16"		B	8"	SYMM
$I_s = 0.0981$						
$M_o = \dfrac{w(\ell_2/2)\ell n^2}{8}$		555 lb.in			555 lb.in	
ΣK_c	0.0187			0.0187		
K_t (1/2 for column line "2")	0.0175			0.0175		
K_{ec}	0.00904			0.00904		
K_s	0.0245		0.0245	0.0245		
$\alpha_{ec} = K_{ec}/\Sigma K_s$	0.369			0.1845		
$1 + \dfrac{1}{\alpha_{ec}}$	3.71			6.4		
I) % M_o (for $\beta_a > 2$)	17.5	55.5	72.3	65	35	
Design M slab	-97	+308	-401	-361	+194	
M for column strip	-97	+185	-301	-271	+116	
M for middle strip	0.00	+123	-100	-90	+78	
II) Provisions for effects of pattern loadings	$\alpha_{min} = 2.1$					
$\alpha_c = \Sigma K_c/\Sigma K_s$	0.76			0.38		
$\delta_s = 1 + \dfrac{2-\beta_a}{4+\beta_a}\left(1 - \dfrac{\alpha_c}{\alpha_{min}}\right)$	1.25			1.31		
		$\dfrac{1.25+1.31}{2}$				
M for column strip		+237			+152	
M for middle strip		+157			+102	

- Longitudinal strips, x-direction

(a) Column lines B, C - Interior:

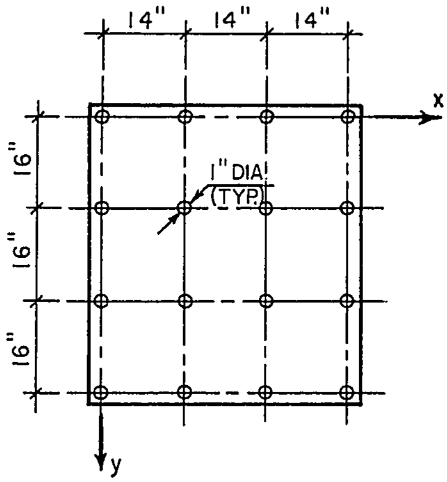
	①	14"		②	7" SYMM
$I_s = 0.153$					
ℓ_1, ℓ_2, c, ℓn, ℓ_2/ℓ_1	14,16,.89,13.11,1.143				
M_o		954.9			954.9
ΣK_c	0.0187			0.0187	
K_t	0.0298			0.0298	
K_{ec}	0.01149			0.01149	
K_s	0.0437		0.0437	0.0437	
α_{ec}	0.263			0.1315	
$1 + \frac{1}{\alpha_{ec}}$	4.8			8.6	
I) % M_o (for $\beta_a > 2$)	13.5	57.2	73	65	35
Design M slab	-129	+546	-697	-621	+334
M for column strip	-129	+328	-523	-466	+200
M for middle strip	0.00	+218	-174	-155	+134
M for column strip/in	-18.4	+46.9	-74.7	-66.6	+28.6
M for middle strip/in	0.00	+24.2	-19.3	-17.2	+14.9
II) Provisions for effects of pattern loadings (ACI 13.3.6)					
$\alpha_{min} = 2.59$					
α_c	0.43			0.215	
δ_s	1.32	$\frac{1.32+1.35}{2} = 1.335$		1.35	
M for column strip	-19	+438	-413	-396	+270
M for middle strip	0.00	+291	-28	-108	+181
M for column strip/in	-2.7	+62.6	-59	-56.6	+38.6
M for middle strip/in	0.00	+32.3	-3.1	-12	+20.1

(b) Column lines A, D - Exterior:

	1	14"		2	7"
$I_s = 0.1077$					
M_o		477.5			477.5
$\Sigma K_c = K_c$(below)	0.0187			0.0187	
K_t	0.0149			0.0149	
K_{ec}	0.0083			0.0083	
K_s	0.03077		0.03077	0.03077	
α_{ec}	0.269			0.135	
$1 + \frac{1}{\alpha_{ec}}$	4.7			8.4	
I) % M_o (for $\beta_a > 2$)	13.8	57	72.9	65	35
Design M slab	-66	+272	-348	-310	+167
M for column strip	-66	+163	-261	-233	+100
M for middle strip	0.0	+109	-87	-77	+67
II) Provisions for effects of pattern loadings	$\alpha_{min} = 2.59$				
$\alpha_c = \frac{\Sigma K_c}{\Sigma K_s}$	0.608			0.304	
δ_s	1.29	$\frac{1.29+1.34}{2} = 1.315$		1.34	
M for column strip		+214			+134
M for middle strip		+143			+90

EXAMPLE "1-B"

Design Moments by ACI 318-63 EMPIRICAL DESIGN METHOD



Design data

- $w = 400$ psf
- No column capitals or drop panels
- Storey height, center to center = 10-1/2"
- Slab thickness = 0.486 in
- $L_x = 14"$ $L_y = 16"$ $L_y/L_x = 1.143$
- $c = 1"$

Empirical design method: limitations are satisfied.

$$M_o = 0.09\ WLF(1 - \frac{2C}{3L})^2$$

in which: $F = 1.15 - C/L$, $W = 14 \times 16 \times \frac{400}{144} = 622.222$ lbs

- Tranverse strips, y-direction:

$F_y = 1.15 - 1/16 = 1.0875$

$M_{oy} = 0.09 \times 622.222 \times 16 \times 1.0875 \left(1 - \frac{2x1}{3x16}\right)^2 = 894.9$ lb.in

Column strip:				
% M_o (-46)	+22	-50	+28	-40
M	+196.9	-447.5	+250.6	-358.0
Middle strip:				
% M_o (-16)	+16	-18	+20	-10
M	+143.2	-161.1	+179.0	-89.5
(Column + Middle) strips:				
% M_o	+38	-68	+48	-50
M	+340.1	-608.6	+429.6	-447.5

- Longitudinal strips, x-direction

$F_x = 1.15 - 1/14 = 1.0788$

$M_{ox} = 0.1 \times 622.222 \times 14 \times 1.0788 \left(1 - \frac{2x1}{3x14}\right)^2 = 767.2$ lb.in

Column strip: (8")				
% M_o (-46)	+22	-50	+28	-40
M	+168.8	-383.6	+214.8	-306.9
% M for 7" (7/8x8")	+19.25	-43.75	+24.5	-35
M for 7" (7/8x8")	+147.7	-335.6	+188.	-268.6
Middle strip: (8")				
% M_o (-16)	+16	-18	+20	-10
M	+122.8	-138.1	+153.5	-76.7
% M for 9" (8" middle + 1" column)	+18.75	-24.25	+23.5	-15
M for 9" (8" middle + 1" column)	+143.8	-186.0	+180.3	-115.1
(col. + mid.) strips 7" 9"				
% M_o	+38	-68	+48	-50
	+291.5	-521.6	+368.3	-383.7

ACI 318-63

Provision 2104(B):

2nd level:

$$I_{c\,min} = \frac{t^3 H}{0.5 + \frac{W_D}{W_L}} \times (2-2.3\ h/H) \qquad \text{assume } W_D = 2\ W_L$$

$H = 10.5/12$ ft , $t = .486$ in.

$\therefore\ I_{c\,min} = 0.04 \times 1.9 = 0.076$

$I_{c\,actual} = .049 \times 1'' = .0491$

$K = .66$

$$R_n = 1 + \frac{(1-K)^2}{2.2(1 + 1.4\ W_D/W_L)} = 1.014 \qquad \text{for -ve moment}$$

$$R_p = 1 + \frac{(1-K)^2}{1.2(1 + 0.1\ W_D/W_L)} = 1.08$$

assume $W_D = 4\ W_L$

$I_{c\,min} = .0287 \times 1.9 = 0.054$

$K = .92$

$R_p = 1 + .004 = 1.004$

assume $W_D = 1$ $\qquad W_L = 3.64$ $\qquad W_D/W_L = .275$

$I_{c\,min} = .246$ $\qquad K = .2$

$R_n = 1.21$

$R_p = 1.52$

APPENDIX D

CHAPTER 13 OF "BUILDING CODE REQUIREMENTS FOR RE-INFORCED CONCRETE (ACI 318-71)", AND CHAPTER 13 OF "COMMENTARY ON BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE (ACI 318-71)"

APPENDIX D

CHAPTER 13. OF THE BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE (ACI 318-71)

SLAB SYSTEM WITH MULTIPLE SQUARE OR RECTANGULAR PANELS

13.0 Notation

c_1 = size of rectangular or equivalent rectangular column, capital, or bracket measured in the direction in which moments are being determined.

c_2 = size of rectangular or equivalent rectangular column, capital, or bracket measured transverse to the direction in which moments are being determined.

C = cross-sectional constant to define the torsional properties. See Eq. (13-7)

d = distance from extreme compression fiber to centroid of tension reinforcement.

E_{cb} = modulus of elasticity for beam concrete.

E_{cc} = modulus of elasticity for column concrete.

E_{cs} = modulus of elasticity for slab concrete.

h = overall thickness of member, in.

I_b = moment of inertia about centroidal axis of gross section of a beam as defined in Section 13.1.5.

I_c = moment of inertia of gross cross section of columns.

I_s = moment of inertia about centroidal axis of gross section of slab

= $h^3/12$ times width of slab specified in definitions of α and β_t.

K_b = flexural stiffness of beam; moment per unit rotation.

K_c = flexural stiffness of column; moment per unit rotation.

K_{cc} = flexural stiffness of an equivalent column; moment per unit rotation. See Eq. (13-5)

K_s = flexural stiffness of slab; moment per unit rotation.

K_t = torsional stiffness of torsional member; moment per unit rotation.

l_c = height of column, center-to-center of floors or roof.

l_n = length of clear span, in the direction moments are being determined, measured face-to-face of supports.

l_1 = length of span in the direction moments are being determined, measured center-to-center of supports.

l_2 = length of span transverse to l_1, measured center-to-center of supports.

M_o = total static design moment.

w = design load per unit area.

w_d = design dead load per unit area.

w_l = design live load per unit area.

x = shorter overall dimension of a rectangular part of a cross section.

y = longer overall dimension of a rectangular part of a cross section.

α = ratio of flexural stiffness of beam section to the flexural stiffness of a width of slab bounded laterally by the center line of the adjacent panel, if any, on each side of the beam.

$$= \frac{E_{cb} I_b}{E_{cs} I_s}$$

α_c = ratio of flexural stiffness of the columns above and below the slab to the combined flexural stiffness of the slabs and beams at a joint taken in the direction moments are being determined.

$$= \frac{\Sigma K_c}{\Sigma (K_s + K_b)}$$

α_{ec} = ratio of flexural stiffness of the equivalent column to the combined flexural stiffness of the slabs and beams at a joint taken in the direction moments are being determined.

$$= \frac{K_{ec}}{\Sigma (K_s + K_b)}$$

α_{min} = minimum α_c to satisfy Section 13.3.6.1(a).

α_1 = α in the direction of l_1.

α_2 = α in the direction of l_2.

β_a = ratio of dead load per unit area to live load per unit area (in each case without load factors).

β_t = a measure of the ratio of torsional stiffness of edge beam section to the flexural stiffness of a width of slab equal to the span length of the beam, center-to-center of supports.

$$= \frac{E_{cb}C}{2E_{cs}I_s}$$

δs = factor defined by Eq. (13-4). See Section 13.3.6.1.

13.1 Scope and definitions

13.1.1 - The provisions of this chapter govern the design of slab systems reinforced for flexure in more than one direction with or without beams between supports. Solid slabs and slabs with recesses or pockets made by permanent or removable fillers between ribs or joists in two directions are included under this definition. Slabs with paneled ceilings are also included under this definition provided the panel of reduced thickness lies entirely within the middle strips, and is at least two-thirds the thickness of the remainder of the slab, exclusive of the drop panel, and is not less than 4 in. thick. The thicknesses shall satisfy requirements of Section 9.5.3.

13.1.2 - A column strip is a design strip with a width of 0.25 l_2 but not greater than 0.25 l_1 on each side of the column center line. The strip includes beams, if any.

13.1.3 - A middle strip is a design strip bounded by two column strips.

13.1.4 - A panel is bounded by column or wall center lines on all sides.

13.1.5 - For monolithic or fully composite construction, the beam includes that portion of the slab on each side of the beam extending a distance equal to the projection of the beam above or below the slab, whichever is greater, but not greater than four times the slab thickness.

13.1.6 - The slab may be supported on walls, columns, or beams. No portion of a column capital shall be considered for structural purposes which lies outside the largest right circular cone or pyramid with a 90 deg vertex which can be included within the outlines of the supporting element.

13.2 Design procedures

13.2.1 - A slab system may be designed by any procedure satisfying the conditions of equilibrium and geometrical compatibility provided it is shown that the strength furnished is at least that required considering Sections 9.2 and 9.3, and that all serviceability conditions, including the specified limits on deflections, are met.

13.2.2 - A slab system, including the slab and any supporting beams, columns, and walls, may be designed directly by either of the procedures described in this chapter: The Direct-Design Method (Section 13.3) or The Equivalent Frame Method (Section 13.4).

13.2.3 - The slabs and beams shall be proportioned for the design bending moments prevailing at every section.

13.2.4 - When unbalanced gravity load, wind, earthquake, or other lateral loads cause transfer of bending moment between slab and column, the flexural stresses shall be investigated using a fraction of the moment given by

$$\frac{1}{1 + 2/3\sqrt{\dfrac{c_1 + d}{c_2 + d}}}$$

A slab width between lines that are one and one-half slab or drop panel thickness, 1.5h, on each side of the column or capital may be considered effective.

Concentration of reinforcement over column head by closer spacing or additional reinforcement may be used to resist the moment on this section.

13.2.5 - Design for the transmission of load from the slab to the supporting walls and columns through shear and torsion shall be in accordance with Chapter 11.

13.3 Direct design method

13.3.1 Limitations

13.3.1.1 There shall be a minimum of three continuous spans in each direction.

13.3.1.2 The panels shall be rectangular with the ratio of the longer to shorter spans within a panel not greater than 2.0.

13.3.1.3 The successive span lengths in each direction shall not differ by more than one-third of the longer span.

13.3.1.4 Columns may be offset a maximum of 10 percent of the span, in direction of the offset, from either axis between center lines of successive columns.

13.3.1.5 All loads shall be due to gravity only and uniformly distributed over an entire panel. The live load shall not exceed three times the dead load.

13.3.1.6 If a panel is supported by beams on all sides, the relative stiffness of the beams in the two perpendicular directions

$$\frac{\alpha_1 \ell_2^{\,2}}{\alpha_2 \ell_1^{\,2}} \qquad (13\text{-}1)$$

shall not be less than 0.2 nor greater than 5.0.

13.3.1.7 Moment redistribution by Section 8.6 shall not be applied for slab systems designed by the direct design method. (See Section 13.3.4.6).

13.3.1.8 Variations from the limitations of this section may be considered acceptable if demonstrated by analysis that the requirements of Section 13.2.1 are satisfied.

13.3.2 Total static design moment for a span

13.3.2.1 The total static design moment for a span shall be determined in a strip bounded laterally by the center line of the panel on each side of the center line of the supports. The absolute sum of the positive and average negative bending moments in each direction shall be not less than

$$M_o = \frac{w l_2 l_n^2}{8} \qquad (13\text{-}2)$$

Where the transverse span of the panels on either side of the center line of supports varies, l_2 shall be taken as the average of the transverse spans. When the span adjacent and parallel to an edge is being considered, the distance from the edge to the panel center line shall be substituted for l_2 in Eq. (13-2).

13.3.2.2 The clear span l_n shall extend from face to face of columns, capitals, brackets, or walls. The value of l_n used in Eq. (13-2) shall be not less than 0.65 l_1. Circular supports shall be treated as square supports having the same area.

13.3.3 Negative and positive design moments

13.3.3.1 The negative design moment shall be located at the face of rectangular supports. Circular supports shall be treated as square supports having the same area.

13.3.3.2 In an interior span, the total static design moment M_o shall be distributed as follows:

Negative design moment	0.65
Positive design moment	0.35

13.3.3.3 In an end span, the total static design moment M_o shall be distributed as follows:

Interior negative design moment	$0.75 - \dfrac{0.10}{1 + \dfrac{1}{\alpha_{ec}}}$
Positive design moment	$0.63 - \dfrac{0.28}{1 + \dfrac{1}{\alpha_{ec}}}$
Exterior negative design moment	$\dfrac{0.65}{1 + \dfrac{1}{\alpha_{ec}}}$

where α_{ec} is computed for the exterior column.

13.3.3.4 The negative moment section shall be designed to resist the larger of the two interior negative design moments determined for the spans framing into a common support unless an analysis is made to distribute the unbalanced moment in accordance with the stiffnesses of the adjoining elements.

13.3.4 Design moments and shears in column and middle strips and beams

13.3.4.1 The column strips shall be proportioned to resist the following portions in percent of the interior negative design moment:

ℓ_2/ℓ_1	0.5	1.0	2.0
$(\alpha_1\ell_2/\ell_1) = 0$	75	75	75
$(\alpha_1\ell_2/\ell_1 \geq 1.0$	90	75	45

Linear interpolations shall be made between the values shown.

13.3.4.2 The column strip shall be proportioned to resist the following portions in percent of the exterior negative design moment:

ℓ_2/ℓ_1		0.5	1.0	2.0
$(\alpha_1\ell_2/\ell_1) = 0$	$\beta_t = 0$	100	100	100
	$\beta_t \geq 2.5$	75	75	75
$(\alpha_1\ell_2/\ell_1) \geq 1.0$	$\beta_t = 0$	100	100	100
	$\beta_t \geq 2.5$	90	75	45

Linear interpolations shall be made between the values shown.

Where the exterior support consists of column or wall extending for a distance equal to or greater than three-quarters of the ℓ_2 used to compute M_o, the exterior negative moment shall be considered to be uniformly distributed across ℓ_2.

13.3.4.3 The column strip shall be proportioned to resist the following portions in percent of the positive design moment:

ℓ_2/ℓ_1	0.5	1.0	2.0
$(\alpha_1\ell_2/\ell_1) = 0$	60	60	60
$(\alpha_1\ell_2/\ell_1 \geq 1.0$	90	75	45

Linear interpolations shall be made between the values shown.

13.3.4.4 The beam shall be proportioned to resist 85 percent of the column strip moment if $(\alpha_1\ell_2/\ell_1)$ is equal to or greater than 1.0. For values of $(\alpha_1\ell_2/\ell_1)$ between 1.0 and zero, the proportion of moment to be resisted by the beam shall be obtained by linear interpolation between 85 and zero percent. Moments caused by loads applied on the beam and not considered in the slab design shall be determined directly. The slab in the column strip shall be proportioned to resist that portion of the design moment not resisted by the beam.

13.3.4.5 That portion of the design moment not resisted by the column strip shall be proportionately assigned to the corresponding half middle strips. Each middle strip shall be proportioned to resist the sum of the moments assigned to its two half middle strips. The middle strip adjacent to and parallel with an edge supported by a wall shall be proportioned to resist twice the moment assigned to the half middle strip corresponding to the first row of interior supports.

13.3.4.6 A design moment may be modified by 10 percent provided the total static design moment for the panel in the direction considered is not less than that required by Eq. (13-2).

13.3.4.7 Beams with $(\alpha_1 \ell_2/\ell_1)$ equal to or greater than 1.0 shall be proportioned to resist the shear caused by loads in tributary areas bounded by 45 deg lines drawn from the corners of the panels and the center line of the panels parallel to the long sides. For values of $(\alpha_1 \ell_2/\ell_1)$ less than 1.0, the shear on the beam may be obtained by linear interpolation, assuming that for $\alpha = 0$ the beams carry no load. In addition, all beams shall be proportioned to resist the shear caused by directly applied loads.

13.3.4.8 The shear stresses in the slab may be computed on the assumption that the load is distributed to the supporting beams in accordance with Section 13.3.4.7. The total shear occurring on the panel shall be accounted for.

13.3.4.9 The shear stresses shall satisfy the requirements of Chapter 11.

13.3.4.10 Edge beams or the edges of the slab shall be proportioned to resist in torsion their share of the exterior negative design moments.

13.3.5 Moments in columns and walls

13.3.5.1 Columns and walls built integrally with the slab system shall resist moments arising from loads on the slab system.

13.3.5.2 At an interior support, the supporting elements above and below the slab shall resist the moment specified by Eq. (13-3) in direct proportion to their stiffnesses unless a general analysis is made.

$$M = \frac{0.08[(w_d+0.5w_\ell)\ell_2\ell_n^2 - w_d'\ell_2'(\ell_n')^2]}{1 + \frac{1}{\alpha_{ec}}} \qquad (13\text{-}3)$$

where w_d', ℓ_2' and ℓ_n' refer to the shorter span.

13.3.6 Provisions for effects of pattern loadings

13.3.6.1 Where the ratio of dead load to live load, β_α, is less than 2.0, one of the following conditions shall be satisfied:

Table 13.3.6.1--Minimum α_{min}

β_a	Aspect ratio ℓ_2/ℓ_1	Relative beam stiffness, α 0	0.5	1.0	2.0	4.0
2.0	0.5-2.0	0	0	0	0	0
1.0	0.5	0.6	0	0	0	0
	0.8	0.7	0	0	0	0
	1.0	0.7	0.1	0	0	0
	1.25	0.8	0.4	0	0	0
	2.0	1.2	0.5	0.2	0	0
0.5	0.5	1.3	0.3	0	0	0
	0.8	1.5	0.5	0.2	0	0
	1.0	1.6	0.6	0.2	0	0
	1.25	1.9	1.0	0.5	0	0
	2.0	4.9	1.6	0.8	0.3	0
0.33	0.5	1.8	0.5	0.1	0	0
	0.8	2.0	0.9	0.3	0	0
	1.0	2.3	0.9	0.4	0	0
	1.25	2.8	1.5	0.8	0.2	0
	2.0	13.0	2.6	1.2	0.5	0.3

(a) The sum of flexural stiffnesses of the columns above and below the slab shall be such that α_c is not less than the minimum α_{min} specified in Table 13.3.6.1.

(b) If the columns do not satisfy (a), the design positive bending moments in the panels supported by those columns shall be multiplied by the coefficient δ_s determined from Eq. (13-4).

$$\delta_s = 1 + \frac{2 - \beta_a}{4 + \beta_a}\left(1 - \frac{\alpha_c}{\alpha_{min}}\right) \quad (13\text{-}4)$$

13.4 Equivalent frame method

13.4.1 Assumptions--In design by the equivalent frame method the following assumptions shall be used and all sections of slabs and supporting members shall be proportioned for the moments and shears thus obtained.

13.4.1.1 The structure shall be considered to be made up of equivalent frames on column lines taken longitudinally and transversely through the building. Each frame consists of a row of equivalent columns or supports and slab-beam strips, bounded laterally by the center line of the panel on each side of the center line of the columns or supports. Frames adjacent and parallel to an edge shall be bounded by the edge and the center line of the adjacent panel.

13.4.1.2 Each such frame may be analyzed in its entirety, or, for vertical loading, each floor thereof and the roof may be analyzed separately with its columns as they occur above and below, the columns being assumed fixed at their remote ends. Where slab-beams are thus analyzed separately, it may be assumed in determining the bending moment at a given support that the slab-beam is fixed at any support two panels distant therefrom provided the slab continues beyond that point.

13.4.1.3 The moment of inertia of the slab-beam or column at any cross section outside of the joint or column capital may be based on the cross-sectional area of the concrete. Variation in the moments of inertia of the slab-beams and columns along their axes shall be taken into account.

13.4.1.4 The moment of inertia of the slab-beam from the center of the column to the face of the column, bracket, or capital shall be assumed equal to the moment of inertia of the slab-beam at the face of the column, bracket, or capital divided by the quantity $(1-c_2/\ell_2)^2$ where c_2 and ℓ_2 are measured transverse to the direction moments are being determined.

13.4.1.5 The equivalent column shall be assumed to consist of the actual columns above and below the slab-beam plus an attached torsional member transverse to the direction in which moments are being determined and extending to the bounding lateral panel center lines on each side of the column. The flexibility (inverse of the stiffness) of the equivalent column shall be taken as the sum of the flexibilities of the columns above and below the slab-beam and the flexibility of the torsional member.

$$\frac{1}{K_{ec}} = \frac{1}{\Sigma K_c} + \frac{1}{K_t} \qquad (13\text{-}5)$$

In computing the stiffness of the column K_c, the moment of inertia shall be assumed infinite from the top to the bottom of the slab-beam at the joint.

The attached torsional members shall be assumed to have a constant cross section throughout their length consisting of the larger of:

(a) A portion of the slab having a width equal to that of the column, bracket, or capital in the direction in which moments are being considered.

(b) For monolithic or fully composite construction, the portion of the slab specified in (a) plus that part of the transverse beam above and below the slab.

(c) The transverse beam as defined in Section 13.1.5.

The stiffness K_t of the torsional member shall be calculated by the following expression:

$$K_t = \Sigma \frac{9E_{cs}C}{\ell_2\left(1 - \frac{c_2}{\ell_2}\right)^3} \qquad (13\text{-}6)$$

where c_2 and ℓ_2 relate to the transverse spans on each side of the column. The constant C in Eq. (13-6) may be evaluated for the cross section by dividing it into separate rectangular parts and carrying out the following summation:

$$C = \Sigma\left(1 - 0.63\,\frac{x}{y}\right)\frac{x^3y}{3} \qquad (13\text{-}7)$$

Where beams frame into the column in the direction moments are being determined, the value of K_t as computed by Eq. (13-6) shall be multiplied by the ratio of the moment of inertia of the slab with such beam to the moment of inertia of the slab without such beam.

13.4.1.6 Where metal column capitals are used, account may be taken of their contributions to stiffness and resistance to bending and to shear.

13.4.1.7 The change in length of columns and slabs due to direct stress, and deflections due to shear, may be neglected.

13.4.1.8 When the loading pattern is known, the structure shall be analyzed for that load. When the live load is variable but does not exceed three-quarters of the dead load, or the nature of the live load is such that all panels will be loaded simultaneously, the maximum moments may be assumed to occur at all sections when full design live load is on the entire slab system. For other conditions, maximum positive moment near midspan of a panel may be assumed to occur when three-quarters of the full design live load is on the panel and on alternate panels; and maximum negative moment in the slab at a support may be assumed to occur when three-quarters of the full design live load is on the adjacent panels only. In no case shall the design moments be taken as less than those occurring with full design live load on all panels.

13.4.2 Negative design moment--At interior supports, the critical section for negative moment, in both the column and middle strips, shall be taken at the face of rectilinear supports, but in no case at a distance greater than $0.175\ell_1$ from the center of the column. At exterior supports provided with brackets or capitals, the critical section for negative moment in the direction perpendicular to the edge shall be taken at a distance from the face of the supporting element not greater than one-half the projection of the bracket or capital beyond the face of the supporting element. Circular or regular polygon shaped supports shall be treated as square supports having the same area.

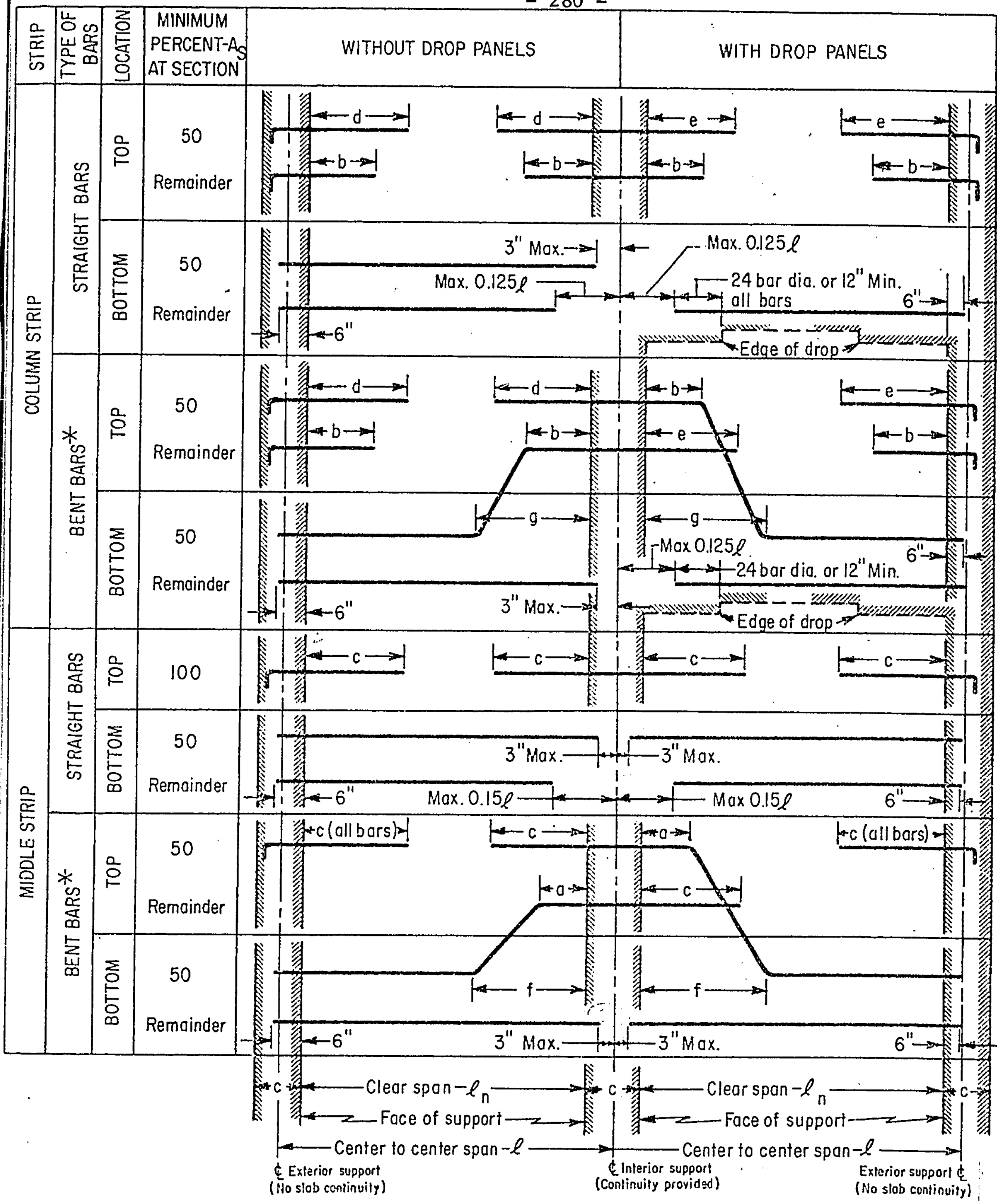
13.4.3 Distribution of panel moments--Bending at critical sections across the slab-beam strip of each frame may be distributed to the column strips, middle strips, and beams as specified in Section 13.3.4 if the requirement of Section 13.3.1.6 is satisfied.

13.4.4 Column moments--Moments determined for the equivalent columns in the frame analysis shall be used in the design of the columns.

13.4.5 Sum of positive and average negative moments--Slabs within the limitations of Section 13.3, when designed by the equivalent frame method, may have the resulting analytical moments reduced in such proportion that the numerical sum of the positive and average negative bending moments used in design need not exceed the value obtained from Eq. (13-2).

13.5 Slab reinforcement

13.5.1 The spacing of the bars at critical sections shall not exceed two times the slab thickness, except for those portions of the slab area which may be of cellular or ribbed construction. In the



* Bent bars at exterior supports may be used if a general analysis is made

	BAR LENGTH FROM FACE OF SUPPORT						
	MINIMUM LENGTH					MAXIMUM LENGTH	
MARK	a	b	c	d	e	f	g
LENGTH	$0.14\ell_n$	$0.20\ell_n$	$0.22\ell_n$	$0.30\ell_n$	$0.33\ell_n$	$0.20\ell_n$	$0.24\ell_n$

Fig. 13.5.6—Minimum bend point locations and extensions for reinforcement in slabs without beams (see Section 12.2.1 for reinforcement extension into supports)

slab over the cellular spaces, reinforcement shall be provided as required by Section 7.13.

13.5.2 In exterior spans, all positive reinforcement perpendicular to the discontinuous edge shall extend to the edge of the slab and have embedment, straight or hooked, of at least 6 in. in spandrel beams, walls, or columns. All negative reinforcement perpendicular to the discontinuous edge shall be bent, hooked, or otherwise anchored, in spandrel beams, walls, or columns, to be developed at the face of the support according to the provisions of Chapter 12. Where the slab is not supported by a spandrel beam or wall, or where the slab cantilevers beyond the support, anchorage of reinforcement may be within the slab.

13.5.3 The area of reinforcement shall be determined from the bending moments at the critical sections but shall not be less than required by Section 7.13.

13.5.4 In slabs supported on beams having a value of α greater than 1.0, special reinforcement shall be provided at exterior corners in both the bottom and top of the slab. This reinforcement shall be provided for a distance in each direction from the corner equal to one-fifth the longer span.

The reinforcement in both the top and bottom of the slab shall be sufficient to resist a moment equal to the maximum positive moment per foot of width in the slab. The direction of the moment is parallel to the diagonal from the corner in the top of the slab and perpendicular to the diagonal in the bottom of the slab. In either the top or bottom of the slab, the reinforcement may be placed in a single band in the direction of the moment or in two bands parallel to the sides of the slab.

13.5.5 Where a drop panel is used to reduce the amount of negative reinforcement over the column of a flat slab, such drop shall extend in each direction, from the center line of support, a distance equal to at least one-sixth the span length measured from center to center of supports in that direction, and the projection below the slab should be at least one-quarter the thickness of the slab beyond the drop. For determining reinforcement, the thickness of the drop panel below the slab shall not be assumed to be more than one-fourth the distance from the edge of the drop panel to the edge of the column capital.

13.5.6 In addition to the other requirements of this section, reinforcement shall have the minimum lengths given in Fig. 13.5.6. Where adjacent spans are unequal, the extension of negative reinforcement beyond the face of the support as prescribed in Fig. 13.5.6 shall be based on the requirements of the longer span.

Bent bars may be used only when the depth-span ratio permits use of bends 45 deg or less. For slabs in frames not braced against sidesway and for slabs resisting lateral loads, slab reinforcement longer than shown in Fig. 13.5.6 shall be provided when required by analysis.

13.6 Openings in the slab system

13.6.1 Openings of any size may be provided in the slab system if it is shown by analysis that the strength furnished is at least that required with consideration of Sections 9.2 and 9.3, and that all serviceability conditions, including the specified limits on deflections, are met.

13.6.2 Openings conforming to the following requirements may be provided in slab systems not having beams without special analysis as required in Section 13.6.1.

(a) Openings of any size may be placed in the area within the middle half of the span in each direction, provided the total amount of reinforcement required for the panel without the opening is maintained.

(b) In the area common to two column strips, not more than one-eighth of the width of strip in either span shall be interrupted by the openings. The equivalent of reinforcement interrupted shall be added on all sides of the openings.

(c) In the area common to one column strip and one middle strip, not more than one-quarter of the reinforcement in either strip shall be interrupted by the opening. The equivalent of reinforcement interrupted shall be added on all sides of the openings.

(d) The shear requirements of Chapter 11 shall be satisfied.

CHAPTER 13. OF COMMENTARY ON BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE (ACI 318-71)

SLAB SYSTEMS WITH MULTIPLE SQUARE OR RECTANGULAR PANELS

In ACI 318-63 and earlier ACI Codes, the design of reinforced concrete slab systems has been handled in different categories because of the history of development of various types of slabs.[13.1] Chapter 13 of ACI 318-71 represents a major change from this practice in that all slab systems reinforced for structural stresses in more than one direction, with or without beams between supports, are designed on the basis of the same fundamental principles. The design methods given in this chapter are based on analysis of the results of an extensive series of tests[13.2-13.9] and the well established performance record of various slab systems.

Much of Chapter 13 is concerned with the selection and distribution of flexural reinforcement. It is, therefore, advisable before discussing the various rules for design, to caution the designer that the vital problem related to safety of the slab system is the transmission of load from the slab to the columns by flexure, torsion, and shear. Design criteria for shear and torsion in slabs are given in Sections 11.10 through 11.13.

13.1 Scope and definitions

13.1.1 The fundamental design principles contained in Chapter 13 are applicable to all planar structural systems subjected to transverse loads. However, some of the specific design rules, as well as historical precedents, limit the types of structures to which Chapter 13 is applicable. General characteristics of slab systems which may be designed according to Chapter 13 are described in this section. These include "flat slabs," "flat plates," "two-way slabs," and "waffle slabs." True "one-way slabs," slabs reinforced to resist flexural stresses in only one direction, are excluded.

Also excluded are soil supported slabs, such as "slabs on grade" which do not transmit vertical loads from other parts of the structure to the soil.

13.1.4 A panel, by definition, includes all flexural elements between column center lines. Thus, the column strip includes the beam, if any.

13.1.5 For monolithic or fully composite construction, the beams include portions of the slab as flanges. Two examples of the rule in this section are provided in Fig. 13-1.

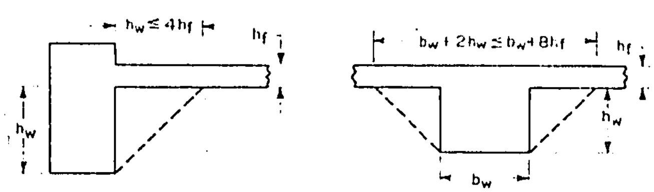


Fig. 13-1—Examples of the portion of slab to be included with the beam under Section 13.1.5

13.2 Design procedures

13.2.1 This section permits a designer to base a design directly on fundamental principles of structural mechanics, provided he can demonstrate explicitly that all safety and serviceability criteria are satisfied. The design of the slab may be achieved through the combined use of classic solutions based on a linearly elastic continuum, numerical solutions based on discrete elements, or yield-line analyses, including, in all cases, evaluation of the stress conditions around the supports in relation to shear and torsion as well as flexure. The designer must consider that the design of a slab system involves more than its analysis, and justify any deviations in physical dimensions of the slab from common practice on the basis of his knowledge of the expected loads and the reliability of the calculated stresses and deformations of the structure.

13.2.2 Two design methods are specified in detail in Chapter 13. The Direct Design Method is similar to the empirical method of the previous Codes, but now applies to slabs with beams as well as to flat slabs and flat plates. Some modifications have been made to the limitations for use of the direct design method (see Section 13.3.1). The Equivalent Frame Method is comparable to the elastic analysis of the previous Codes and also is applicable to slabs with and without beams.

13.2.4 This section is concerned primarily with slab systems without beams. Tests and experience have shown that, unless special measures are taken to resist the torsional and shear stresses, all reinforcement resisting that part of the moment to be transferred to the column by flexure should be placed between lines that are one and one-half the slab or drop panel thickness, 1.5h, on each side of the column. The calculated shear stresses in the slab around the column must conform to the requirements of Sections 11.10, 11.11, 11.12, and 11.13.

13.3 Direct design method

The direct design method consists of a set of rules for the proportioning of slab and beam sections to resist flexural stresses. The rules have been developed to satisfy the safety requirements and most of the serviceability requirements simultaneously. Methodologically, the direct design method compares with the "empirical method" for the flat slabs included in preceding editions of the ACI Code. However, the range of applicability of the method has been much extended in relation to the "empirical method."

The direct design method involves three fundamental steps, as follows:

1. Determination of the total design moment (Section 13.3.2)
2. Distribution of the total design moment to design sections for negative and positive moment (Section 13.3.3)
3. Distribution of the negative and positive design moments to the column and middle strips and to the beams, if any (Section 13.3.4)

13.3.1 Limitations - The direct design method was developed from considerations of theoretical procedures for the determination of moments in slabs with and without beams, requirements for simple design and construction procedures, and precedents supplied by performance of slab systems. Consequently, the slab system to be designed using the direct design method must conform to the limitations in this section.

13.3.1.1 The primary reason for the limitation in this section is the magnitude of the negative moments at the interior support in a structure with only two continuous spans. The rules given for the direct design method assume tacitly that the slab system at the first interior negative moment section is neither fixed against rotation nor discontinuous.

13.3.1.2 If the ratio of the two spans (long span/short span) of a panel exceeds two, the slab resists the moment in the shorter span essentially as a one-way slab. The limiting ratio has been increased from the 1.33 limit stipulated for the "empirical method" of the 1963 Code.

13.3.1.3 The limitation in this section is related to the possibility of developing negative moments at or near midspan. The limiting variation in span lengths has been increased from that for the 1963 "empirical method."

13.3.1.4 In keeping with previous practice, the designer is permitted to offset the columns within specified limits from a regular rectangular array.

13.3.1.5 This section relates to the effect of pattern loads. No limitation of ratio of live load to dead load was placed on the use of the "empirical method" in 1963, but as stated above, the limitations on span ratios and length to width ratios have been relaxed.

The Direct Design Method is based on tests for uniform gravity loads and resulting support (columns) reactions determined by statics. Lateral loads (wind, seismic, etc.) require a frame analysis. Inverted foundation mats designed as two-way slabs (Section 15.10) involve application of known column loads. Therefore, even where the soil reaction is assumed to be uniform, a frame analysis is required.

13.3.1.6 The elastic distribution of moments will deviate significantly from those assumed in the direct design method unless the given requirements for stiffness are satisfied.

13.3.1.7 Moment redistribution by Section 8.6 is not intended where approximate values for bending moments are used. For the direct design method, only a 10 percent modification is allowed by Section 13.3.4.6.

13.3.1.8 The designer is permitted to use the direct design method even if the structure does not fit the limitations in this section, provided that he can show by analysis that the particular limitation does not apply to that structure. For example, in the case of a slab system carrying a nonmovable load (such as a water reservoir in which the load on all panels is expected to be the same), the designer does not have to satisfy Section 13.3.1.5.

13.3.2 Total static design moment for a span

13.3.2.1 Eq. (13.2) follows directly from Nichol's derivation[13.10] with the simplifying assumption that the reactions are concentrated along the faces of the support perpendicular to the span considered. In general, the designer will find it expedient to calculate static moments for two adjacent half panels, which include a column strip with a half middle strip along each side as shown in Fig. 13-2.

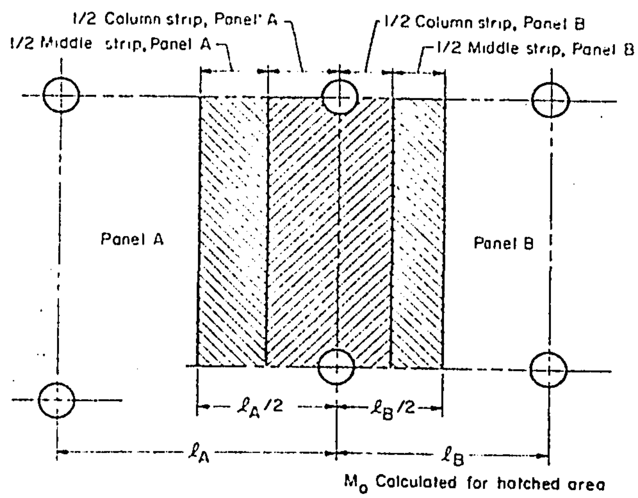


Fig. 13-2—Suggested area to be considered in calculating static moments under Section 13.3.2.1

13.3.2.2. If the calculated value of ℓ_n is less than $0.65\ell_1$, the span should be considered as being $0.65\ell_1$. If a supporting member does not have a rectangular cross section, it is to be treated as a square support having the same area, as illustrated in Fig. 13-3.

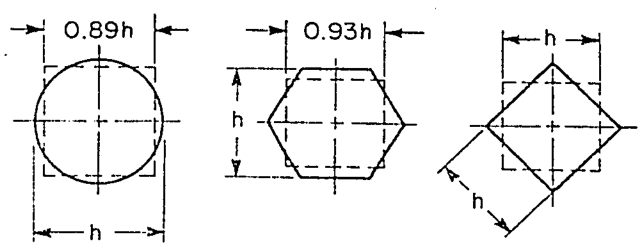


Fig. 13-3—Examples of equivalent square section for nonrectangular supporting members

13.3.3.Negative and positive design moments - The rules for assigning the total design moment to the negative and positive design moments are summarized in Fig. 13-4. The proportions are based on three-dimensional analytical studies of elastic distribution of moments in various slab configurations tempered by the distributions of moments that have been in use.

NEGATIVE AND POSITIVE DESIGN MOMENTS

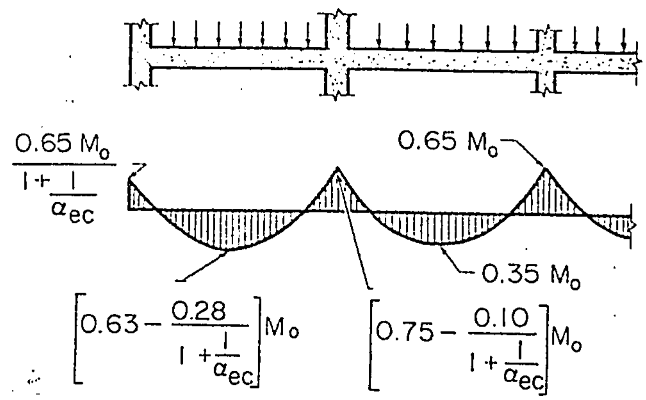


Fig. 13-4—Summary of rules for dividing the total static moment or the total design moment into negative and positive design moments

13.3.3.3 The term α_{ec} in the equations of this section is the flexural stiffness of an equivalent exterior column K_{ec} relative to the flexural stiffness of the slab and the beams, if any. The calculation of K_{ec} is described in Section 13.4.1.5, and rules are given for the calculation of K_c and K_s in Sections 13.4.1.3 and 13.4.1.4, all in connection with the equivalent frame method. Since the use of the direct design method is limited by the requirements of Section 13.3.1, it is permissible to make certain simplifications in the calculations of α_{ec} for use in Section 13.3.3.3, as follows:

1. The stiffness of the slab-beam K_s may be calculated using a uniform cross section between column center lines, disregarding the

requirement of Section 13.4.1.4, and the increase in column stiffness K_c provided by the capital may be neglected, disregarding the requirement of Section 13.4.1.3. Since both of these simplifications lead to less stiff elements, one should not be made without the other in order to minimize the change in relative stiffness.

2. The requirement in the last sentence of Section 13.4.1.5 may be waived in calculations of α_{ec} for use with the direct design method.

3. If a corner column is the same size as an adjacent exterior column, it will be acceptable to use for the corner column the value of α_{ec} computed for the adjacent exterior column. This approximation will usually lead to somewhat smaller exterior negative design moments and somewhat larger positive and interior negative design moments than the more rigorous analysis required for the equivalent frame method.

The detailing of the reinforcement transferring the moment from the slab to the exterior column is critical to both the performance and the safety of flat slabs without edge beams or equivalent cantilever slabs. This reinforcement must be placed in accordance with Section 13.2.4.

13.3.3.4 The differences in slab moment on either side of a column or other type of support must be accounted for in the design of the support. If an analysis is made to distribute unbalanced moments, flexural stiffness may be obtained on the basis of the gross plain concrete section of the members involved.

13.3.4 Design moments and shears on column and middle strips and beams - The rules given in this section for assigning moments to the middle strip, column strip, and beams, if any, are based on studies of moments in linearly elastic slabs with different beam stiffnesses[13.11] tempered by the moment coefficients that have been used successfully in the past.

Where walls are used as supports along column lines, they can be regarded as very stiff beams with an $\alpha_1 \ell_2/\ell_1$, value greater than one. Where the exterior support consists of a wall perpendicular to the direction in which moments are being determined, β_t may be taken as zero if the wall is of masonry without torsional resistance, and β_t may be taken as 2.5 for a concrete wall with great torsional resistance which is monolithic with the slab.

For the purpose of establishing moments in the half column strip adjacent to an edge supported by a wall, ℓ_n in Eq. (13-2) may be assumed equal to ℓ_n of the parallel adjacent column to column span, and the wall may be considered as a beam having a moment of inertia, I_b, equal to infinity.

13.3.4.2 The effect of the torsional stiffness parameter β_t is to assign all of the exterior negative design moment to the column strip, and none to the middle strip, unless the beam torsional stiffness is high relative to the flexural stiffness of the supported slab. In the definition of β_t, the shear modulus has been taken as $E_{cb}/2$.

13.3.4.7 and 13.3.4.8 The tributary area for the shear on an interior beam is shown shaded in Fig. 13-5. If the stiffness for the

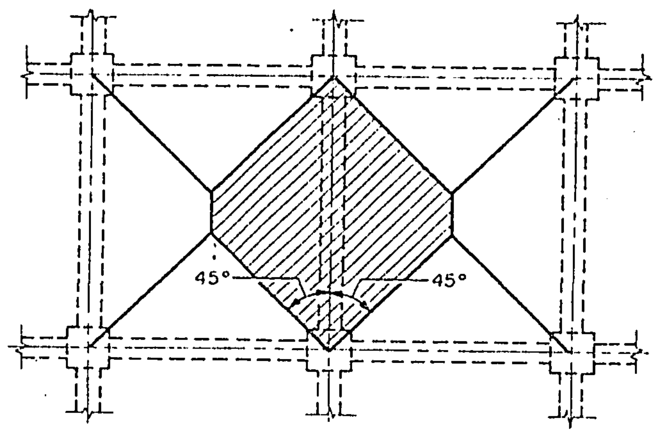


Fig. 13-5—Tributary area for shear on an interior beam

beam $\alpha_1 \ell_2/\ell_1$ is less than one, the shear on the beam may be obtained by linear interpolation. For such cases, the beams framing into the column will not account for all the shear force applied on the column. The remaining shear force will produce shear stresses in the slab around the column which must be checked in the same manner as for flat slabs, as required by Section 13.3.4.8. Section 13.3.4.7 does not apply to the calculation of torsional moments on the beams. These moments must be based on the calculated flexural moments acting on the sides of the beam.

13.3.4.10 Design moments perpendicular to, and at the edge of, the slab structure must be transmitted to the supporting columns or walls. Torsional stresses caused by the moment assigned to the slab should be investigated.

13.3.5 Moments in columns and walls

13.3.5.2 Eq. (13-3) refers to two adjoining spans, with one span longer than the other, the full load applied on the longer span and only the dead load applied on the shorter span. The term α_{ec} refers to the flexural stiffness of the columns between the two spans.

In the calculation of α_{ec} it is permissible to make the simplifications given as Items (1) and (2) in the commentary on Section 13.3.3.3. In addition, when applying Eq. (13-3) to determine the moment in an exterior column in a direction parallel to the edge of the panel, it is conservative, and therefore permissible, to use in Eq. (13-3) the value of α_{ec} computed for the adjacent interior column, provided the columns are of the same size.

13.3.6 Provisions for effects of pattern loading - The requirements in this section limit the possible increases in moment as a result of pattern loadings at the service load level and are based on analytical and experimental studies summarized in Reference 13.12. Values of the column flexural stiffness α_{min}, required to limit the increase in bending moment caused by pattern loads in less than 33 percent, are listed in Table 13.3.6.1 of the Code as a ratio of the flexural stiffness of the slab system. If the columns of a particular structure do not satisfy the required values of α_{min}, the positive moment in the slab must be increased in accordance with Eq. (13-4).

When applying Eq. (13-4) to moments in the half column strip parallel to an exterior panel edge, it is conservative, and therefore permissible, to use in Eq. (13-4) the value of α_c computed for the adjacent interior column, provided the columns are of the same size.

13.4 Equivalent frame method

The equivalent frame method involves the representation of the three-dimensional slab system by a series of two-dimensional frames which are then analyzed for loads, either vertical or horizontal, acting in the plane of the frames. The moments so determined at the critical design sections of the frame are distributed to the slab sections in accordance with Section 13.3.3.

The equivalent frame method is comparable to the "elastic analysis" for flat slabs of previous ACI Codes. On the basis of studies reported in References 13.13, 13.14, and 13.15, the equivalent frame method has been devised to provide a better representation in two dimensions of a three-dimensional system through the stratagem of defining flexural stiffnesses which reflect the torsional rotations possible in the three-dimensional system.

13.4.1.1 The application of the requirements of this section to a regular structure is illustrated in Fig. 13-6. The shaded areas represent the extent of one interior and one exterior frame in a given direction. See also Reference 13.15.

13.4.1.2-13.4.1.7 These sections permit simplifications which may be used in analyzing the frame. The use of a high-speed computer may make it easy to use a comprehensive analysis without resorting to these simplifications.

13.4.1.4 This section stipulates a finite moment of inertia for the slab-beam from the face of the column or capital to the center

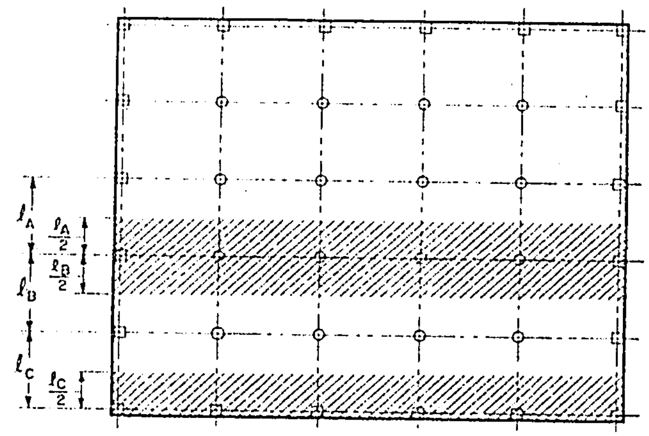
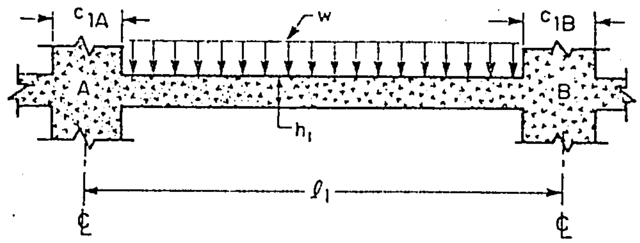


Fig. 13-6—Area to be considered as equivalent frame under Section 13.4

line of the support to account for the flexibility of the slab on the "sides" of the column in that portion of the span. Fixed-end moments for uniform load, stiffness factors, and carryover factors for slabs without beams and having a varying moment of inertia in accordance with Section 13.4.1.4 are listed in Tables 13-1 and 13-2 (Reference 13.16).

13.4.1.5 This section modifies the column flexural stiffness to account for the torsional flexibility of the slab. The intent of this section is illustrated by the simplified physical model in Fig. 13-7 which represents a column AB, extending above and below the slab, with a portion of the slab CD attached thereto. A moment M applied along CD will cause a torsional rotation of the "cross beam" CD as well as a flexural rotation of the column. Thus, the rotational restraint on the slab-beam which spans in a direction perpendicular

TABLE 13-1—MOMENT DISTRIBUTION CONSTANTS*

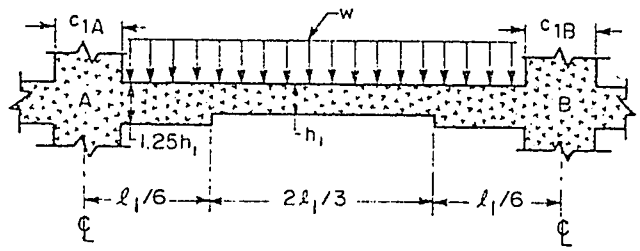


Column dimension		Uniform load FEM = Coef. ($wl_2l_1^2$)		Stiffness factor†		Carryover factor	
$\frac{c_{1A}}{l_1}$	$\frac{c_{1B}}{l_1}$	M_{AB}	M_{BA}	k_{AB}	k_{BA}	COF_{AB}	COF_{BA}
	0.00	0.083	0.083	4.00	4.00	0.500	0.500
	0.05	0.083	0.084	4.01	4.04	0.504	0.500
	0.10	0.082	0.086	4.03	4.15	0.513	0.499
0.00	0.15	0.081	0.089	4.07	4.32	0.528	0.498
	0.20	0.079	0.093	4.12	4.56	0.548	0.495
	0.25	0.077	0.097	4.18	4.88	0.573	0.491
	0.30	0.075	0.102	4.25	5.28	0.603	0.485
	0.35	0.073	0.107	4.33	5.78	0.638	0.478
	0.05	0.084	0.084	4.05	4.05	0.503	0.503
	0.10	0.083	0.086	4.07	4.15	0.513	0.503
	0.15	0.081	0.089	4.11	4.33	0.528	0.501
0.05	0.20	0.080	0.092	4.16	4.58	0.548	0.499
	0.25	0.078	0.096	4.22	4.89	0.573	0.494
	0.30	0.076	0.101	4.29	5.30	0.603	0.489
	0.35	0.074	0.107	4.37	5.80	0.638	0.481
	0.10	0.085	0.085	4.18	4.18	0.513	0.513
	0.15	0.083	0.088	4.22	4.36	0.528	0.511
0.10	0.20	0.082	0.091	4.27	4.61	0.548	0.508
	0.25	0.080	0.095	4.34	4.93	0.573	0.504
	0.30	0.078	0.100	4.41	5.34	0.602	0.498
	0.35	0.075	0.105	4.50	5.85	0.637	0.491
	0.15	0.086	0.086	4.40	4.40	0.526	0.526
	0.20	0.084	0.090	4.46	4.65	0.546	0.523
0.15	0.25	0.083	0.094	4.53	4.98	0.571	0.519
	0.30	0.080	0.099	4.61	5.40	0.601	0.513
	0.35	0.078	0.104	4.70	5.92	0.635	0.505
	0.20	0.088	0.088	4.72	4.72	0.543	0.543
0.20	0.25	0.086	0.092	4.79	5.05	0.568	0.539
	0.30	0.083	0.097	4.88	5.48	0.597	0.532
	0.35	0.081	0.102	4.99	6.01	0.632	0.524
	0.25	0.090	0.090	5.14	5.14	0.563	0.563
0.25	0.30	0.088	0.095	5.24	5.58	0.592	0.556
	0.35	0.085	0.100	5.36	6.12	0.626	0.548
0.30	0.30	0.092	0.092	5.69	5.69	0.585	0.585
	0.35	0.090	0.097	5.83	6.26	0.619	0.576
0.35	0.35	0.095	0.095	6.42	6.42	0.609	0.609

*Applicable when $c_1/l_1 = c_2/l_2$. For other relationships between these ratios, the constants will be slightly in error.

†Stiffness is, $K_{AB} = k_{AB}E\frac{l_2h_1^3}{12l_1}$, and $K_{BA} = k_{BA}E\frac{l_2h_1^3}{12l_1}$.

TABLE 13-2—MOMENT DISTRIBUTION CONSTANTS*



Column dimension		Uniform load FEM = Coef. ($wl_2l_1^2$)		Stiffness factor†		Carryover factor	
$\frac{c_{1A}}{l_1}$	$\frac{c_{1B}}{l_1}$	M_{AB}	M_{BA}	k_{AB}	k_{BA}	COF_{AB}	COF_{BA}
0.00	0.00	0.088	0.088	4.78	4.78	0.541	0.541
	0.05	0.087	0.089	4.80	4.82	0.545	0.541
	0.10	0.087	0.090	4.83	4.94	0.553	0.541
	0.15	0.085	0.093	4.87	5.12	0.567	0.540
	0.20	0.084	0.096	4.93	5.36	0.585	0.537
	0.25	0.082	0.100	5.00	5.68	0.606	0.534
	0.30	0.080	0.105	5.09	6.07	0.631	0.529
0.05	0.05	0.088	0.088	4.84	4.84	0.545	0.545
	0.10	0.087	0.090	4.87	4.95	0.553	0.544
	0.15	0.085	0.093	4.91	5.13	0.567	0.543
	0.20	0.084	0.096	4.97	5.38	0.584	0.541
	0.25	0.082	0.100	5.05	5.70	0.606	0.537
	0.30	0.080	0.104	5.13	6.09	0.632	0.532
0.10	0.10	0.089	0.089	4.98	4.98	0.553	0.553
	0.15	0.088	0.092	5.03	5.16	0.566	0.551
	0.20	0.086	0.094	5.09	5.42	0.584	0.549
	0.25	0.084	0.099	5.17	5.74	0.606	0.546
	0.30	0.082	0.103	5.26	6.13	0.631	0.541
0.15	0.15	0.090	0.090	5.22	5.22	0.565	0.565
	0.20	0.089	0.094	5.28	5.47	0.583	0.563
	0.25	0.087	0.097	5.37	5.80	0.604	0.559
	0.30	0.085	0.102	5.46	6.21	0.630	0.554
0.20	0.20	0.092	0.092	5.55	5.55	0.580	0.580
	0.25	0.090	0.096	5.64	5.88	0.602	0.577
	0.30	0.088	0.100	5.74	6.30	0.627	0.571
0.25	0.25	0.094	0.094	5.98	5.98	0.598	0.598
	0.30	0.091	0.098	6.10	6.41	0.622	0.593
0.30	0.30	0.095	0.095	6.54	6.54	0.617	0.617

*Applicable when $c_1/l_1 = c_2/l_2$. For other relationships between these ratios, the constants will be slightly in error.

†Stiffness is $K_{AB} = k_{AB}E\,\frac{l_2h_1^3}{12l_1}$, and $K_{BA} = k_{BA}E\,\frac{l_2h_1^3}{12l_1}$.

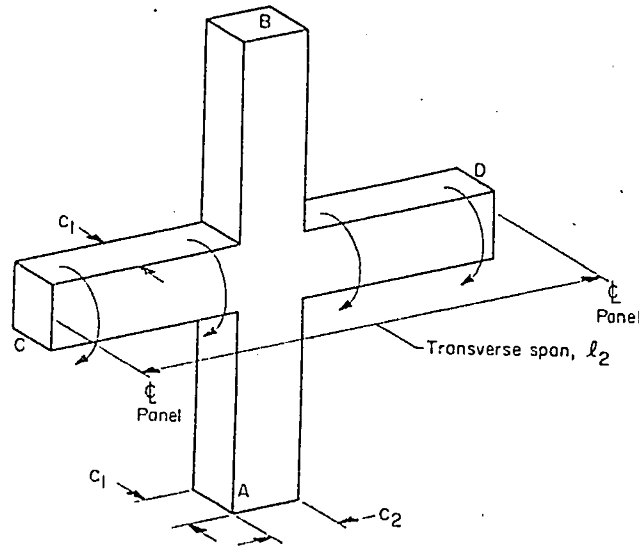


Fig. 13-7—Simplified physical model illustrating the intent of Section 13.4.1.5

to AB and CD, depends on both the torsional rotation of CD and the flexural rotation of AB.

The overall flexibility, $1/K_{ec}$, of the composite element shown in Fig. 13-7 is assumed to be the sum of the flexibility of the columns, $1/\Sigma K_c$, and the torsional flexibility of the "beam", $1/K_t$, as given in Eq. (13-5) in the Code.

The stiffness K_c is based on the length of the column from middepth of slab above to middepth of slab below and its moment of inertia, which is computed on the basis of its cross section, taking into account the increase in stiffness provided by the capital, if any. The column is assumed to be infinitely stiff over the depth of the slab.

The increase in column stiffness provided by the capital may be disregarded when using the direct design method.

Where the exterior edge of a slab system is supported on a concrete wall monolithic with the slab, the flexibility of the wall should replace the column flexibility $1/K_c$ in Eq. (13-5) and the torsional flexibility $1/K_t$ of the wall may be assumed to be zero in the same equation. Where the exterior edge of a slab system is supported on an unreinforced masonry wall, K_{ec} may be taken as zero.

The computation of the torsional stiffness K_t requires several simplifying assumptions. If no beam frames into the column, a portion of the slab equal to the width of the column or capital is assumed as the effective beam. If a beam frames into the column, T-beam or L-beam action is assumed, with the flanges extending on each side of the beam a distance equal to the projection of the beam above or below the slab but not greater than four times the thickness of the slab. Furthermore, it is assumed that no torsional rotation occurs in the beam over the width of the support.

Studies of three-dimensional analyses of various slab configurations suggest that a reasonable value of the torsional stiffness can be obtained by assuming a moment distribution along the beam CD in Fig. 13-7 which varies linearly from a maximum at the center of the column to zero at the middle of the panel. The assumed distribution of unit twisting moment along the column center line, the twisting moment diagram, and the resulting unit rotation diagram are shown in Fig. 13-8.

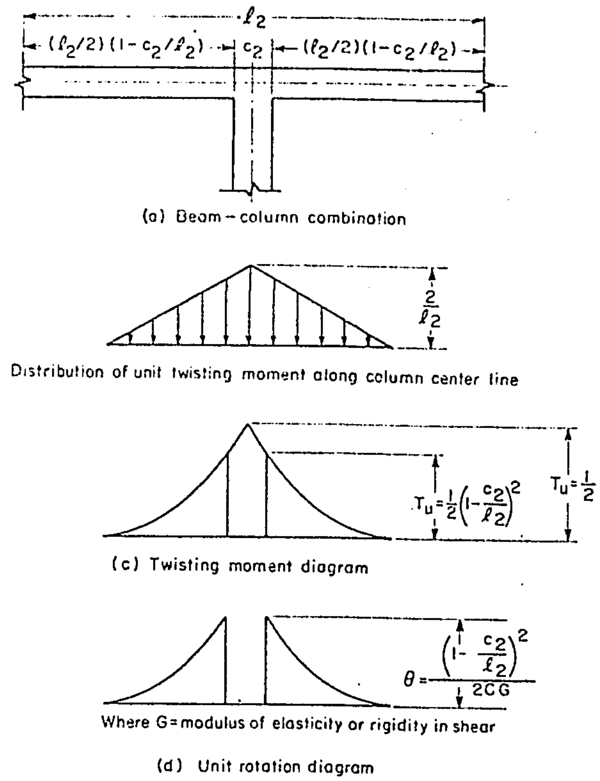


Fig. 13-8—Assumed distribution of unit twisting moment along column center line, twisting moment diagram, and unit rotation diagram

Eq. (13-6) is an approximate expression for the stiffness of the torsional member, based on the results of three-dimensional analyses of various slab configurations. The development of this expression and the assumptions on which it is based, as well as the justification for its use, are discussed in References 13.13, 13.14, and 13.15.

The term C is a property of the cross section having the same relationship to the torsional rigidity of a noncircular cross section as does the polar moment of inertia for a circular cross section. The term

$$(1 - 0.63 \frac{x}{y}) \frac{x^3 y}{3}$$

in Eq. (13-7) is a conservatively low approximation to the value of C for a rectangular section, assuming elastic behavior (see for example Reference 13.16). Since the value of C obtained by summing the values for each of the component rectangles making up a section will always be less than the theoretically correct value, it is appropriate to subdivide the cross section in such a way as to result in the highest possible value of C.

If a panel contains a beam parallel to the direction in which moments are being determined, the value of K_t obtained from Eq. (13-6) may lead to equivalent column stiffnesses which are too low. In such cases, the value of K_t given by Eq. (13-6) should be increased as follows:

$$K_{ta} = K_t \frac{I_{sb}}{I_s}$$

where K_{ta} is the increased torsional stiffness due to presence of parallel beam, I_s is the moment of inertia of a width of slab equal to the full width between center lines of panels, not considering the parallel beam, and I_{sb} is the moment of inertia of the width of slab used for the calculation of I_s, but including the contribution of that portion of the beam stem extending above or below the slab; the beam as defined in Section 13.1.5 does not apply in this calculation.

13.4.1.8 The use of only three-quarters of the full design live load for maximum-moment loading patterns is based on the fact that maximum negative and maximum positive live load moments cannot occur simultaneously and that redistribution of maximum moments is thus possible before failure occurs. This procedure, in effect, permits some local overstress under the full design live load if it is distributed in the prescribed manner, but still insures that the ultimate capacity of the slab system after redistribution of moment is not less than that required to carry the full design dead and live loads on all panels.

13.4.2 This section corrects the negative design moments to the face of the supports. The correction is modified at an exterior support in order not to result in undue reductions in the exterior negative moment. Fig. 13-3 illustrates several equivalent rectangular supports for use in establishing faces of supports for design with nonrectangular supports.

13.4.5 This section is a holdover from many previous Codes and is based on the principle that if two different methods are prescribed to obtain a particular answer, the Code should not require a value greater than the least acceptable value. Due to the long satisfactory experience with designs having total static design moments not exceeding those given by Eq. (13-2), it is considered that these values are satisfactory for design.

13.5 Slab reinforcement

The requirement that the center to center spacing of reinforcement be not more than two times the slab thickness applies only to the reinforcement in solid slabs, and not to that in joists or waffle slabs. This limitation is intended to insure slab action and reduce cracking and to provide for the possibility of loads concentrated on small areas of the slab.

13.5.2 Bending moments in slabs at spandrel beams can be subject to great variation. If the beam should be built solidly into a masonry wall, the slab could be fixed. If no wall, the slab could be largely simply supported. This regulation provides for unknown conditions that might normally occur in a structure.

13.5.6 Where two-way slabs act as primary members of a laterally unbraced frame resisting lateral loads, the resulting moments due to the combined lateral and gravity loadings preclude use of the arbitrary minimum and maximum bends and minimum extensions of bars in Fig. 13.5.6. The bent bar system is applicable for middle strips where $(d - d' + d_b) \leqq 0.06\ \ell_n$ and for column strips where $(d - d' + d_b) \leqq 0.04\ \ell_n$ which permits a bend angle of 45 deg or less.

13.6 Openings in the slab system

See Commentary for Section 11.12 and Fig. 11-8.

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APPENDIX E

DEFLECTION DIAGRAMS

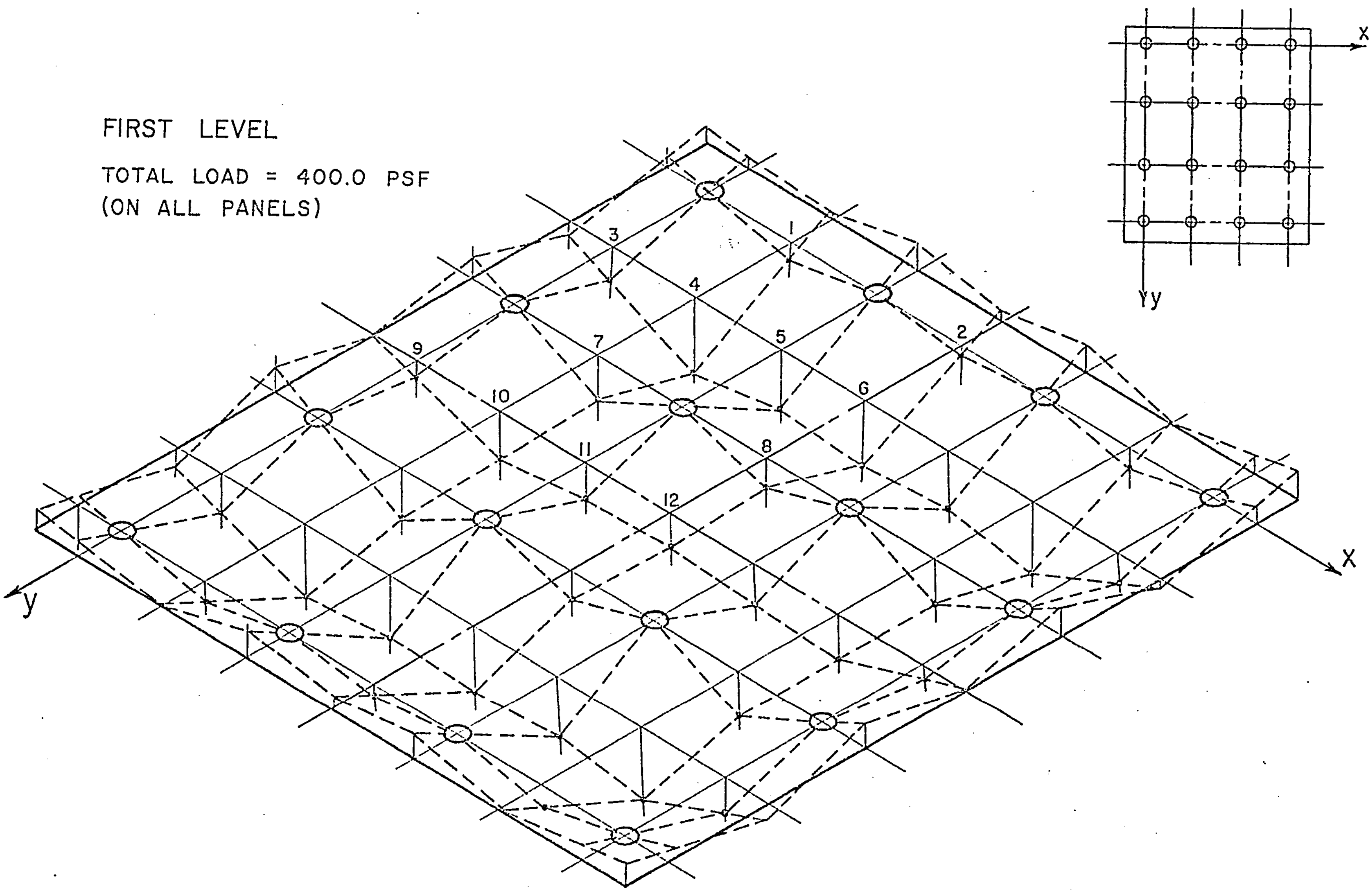

FIG. E1 : SCHEMATIC DEFLECTION DIAGRAM.

FIRST LEVEL

TOTAL LOAD = 400.0 PSF
(ON ALL PANELS)

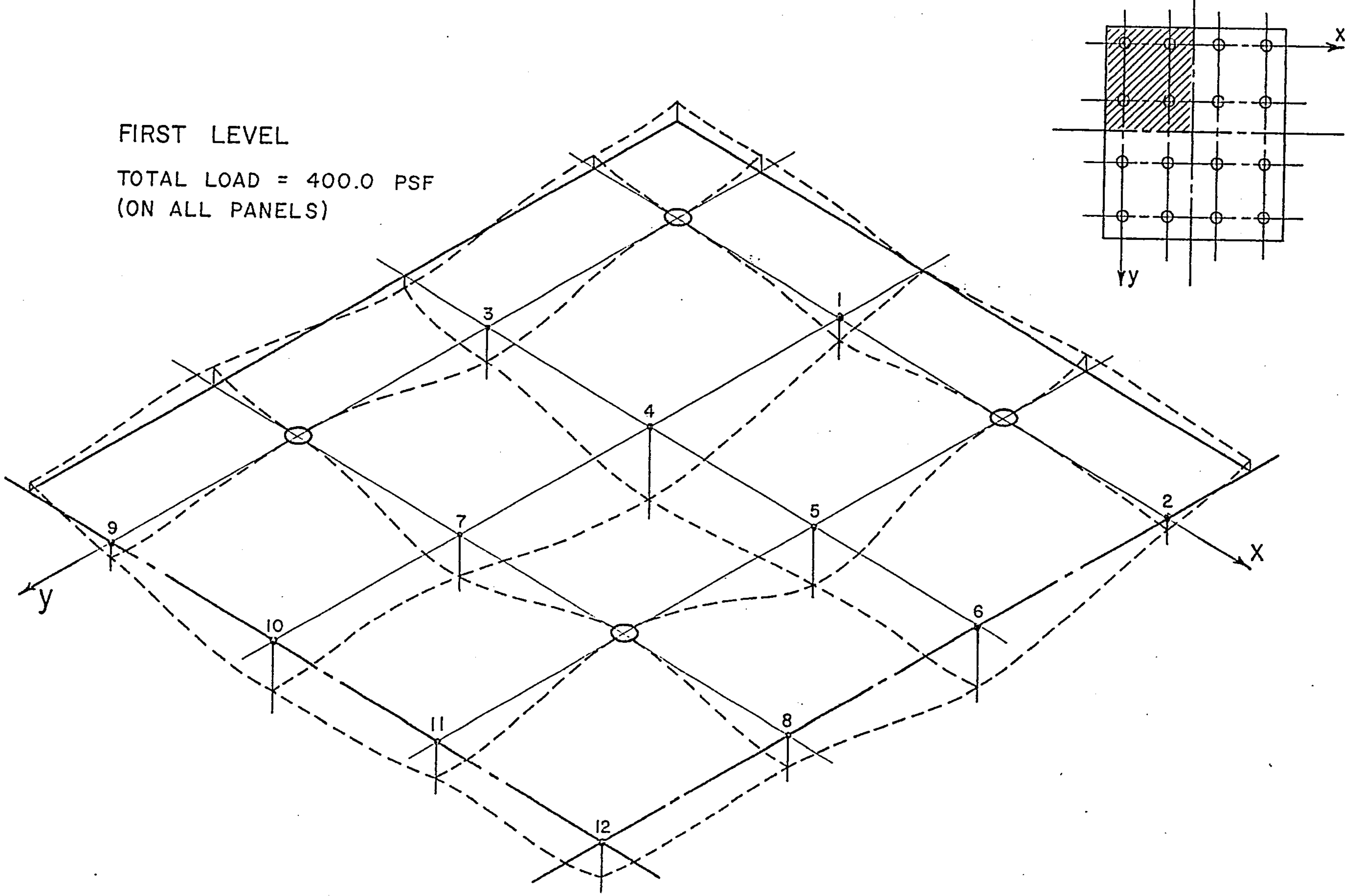

FIG. E.2 : SCHEMATIC DEFLECTION DIAGRAM.
(FOR ONE QUARTER OF THE FIRST LEVEL SLAB)

SECOND LEVEL

TOTAL LOAD = 400.0 PSF
(ON ALL PANELS)

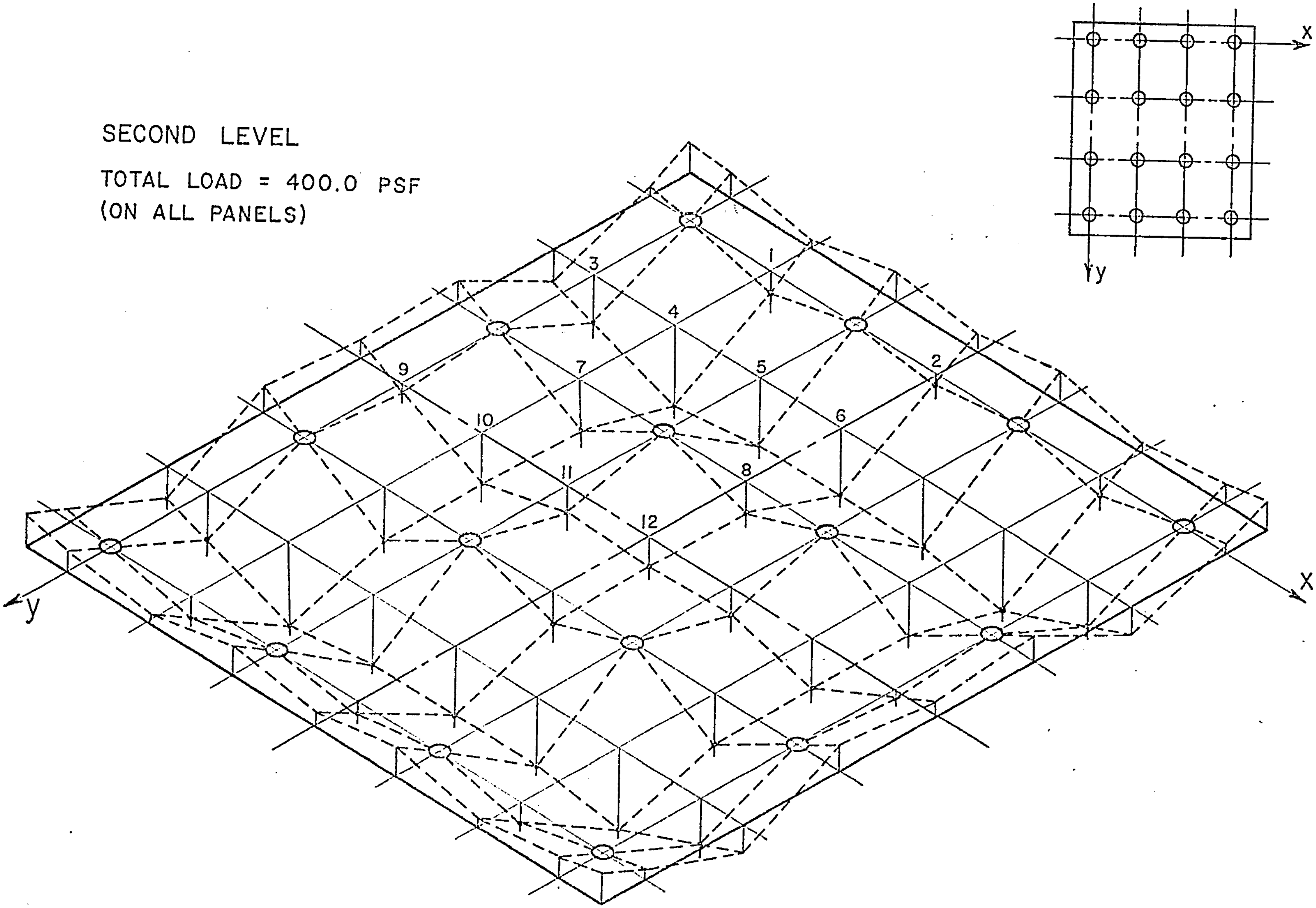

FIG. E3 : SCHEMATIC DEFLECTION DIAGRAM.

SECOND LEVEL
TOTAL LOAD = 400.0 PSF
(ON ALL PANELS)

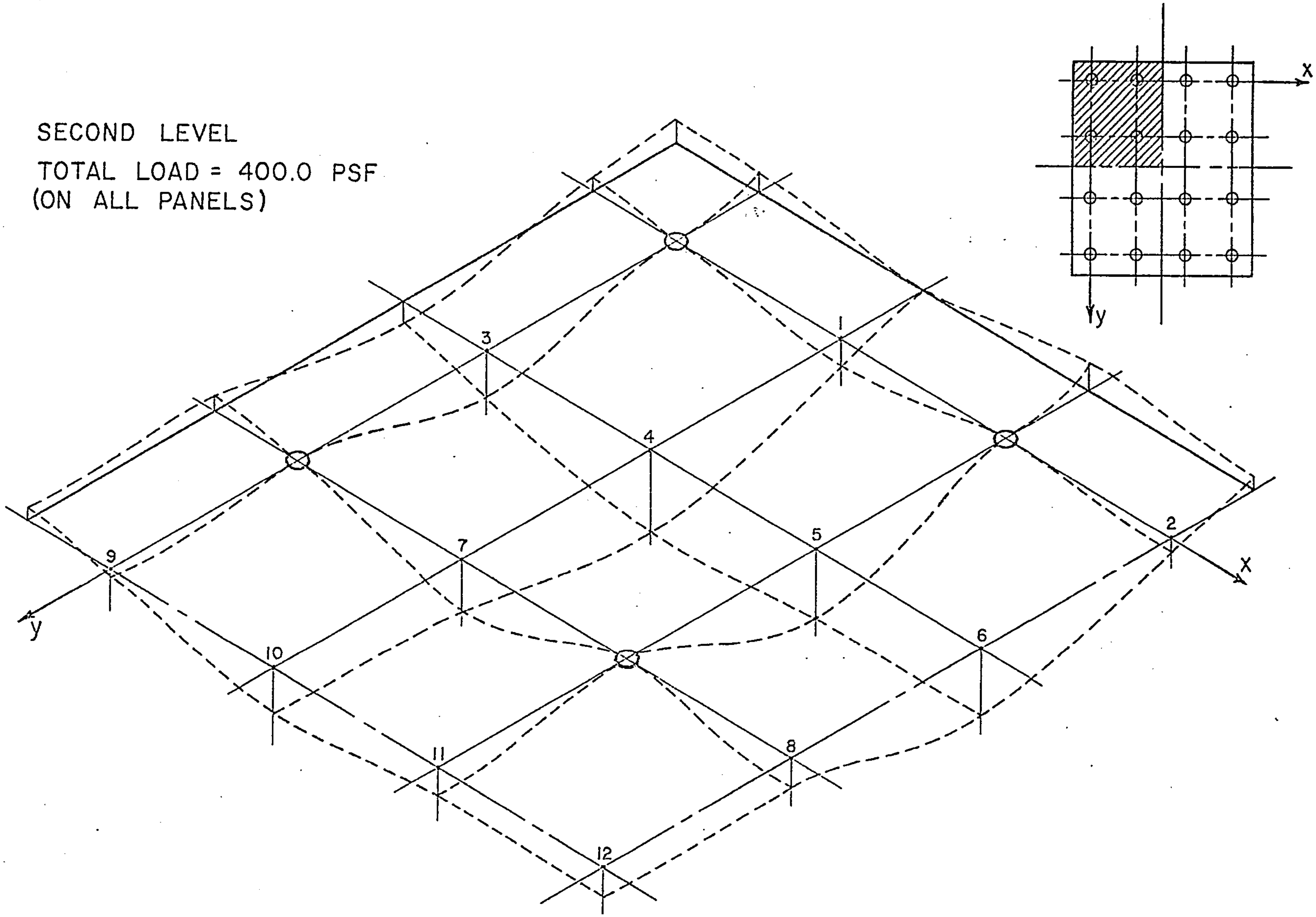

FIG. E 4: SCHEMATIC DEFLECTION DIAGRAM
(FOR ONE QUARTER OF THE SECOND LEVEL SLAB)