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FRAME-FLOOR-WALL SYSTEM INTERACTION IN BUILDINGS

by

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ABSTRACT

A major consideration in the design of steel framed multistory buildings is the limitation of drift at working loads to an acceptable value. For buildings over about twenty stories and employing unbraced steel frame construction drift will likely be the controlling design criterion rather than strength. Traditionally, drift is controlled by increasing the size of frame members, usually the beams. The stiffening effect arising from the interaction of the frames with the floor and wall or cladding systems is usually ignored or crudely estimated. The primary purpose of this investigation is to study these interactions both analytically and experimentally and to suggest simple methods whereby structural interactions may be considered in the design. The study was limited to interactions between unbraced rectangular steel frames, composite steel-concrete floor systems and light-gage corrugated or rib type vertical structural partitions. The study concluded that the major interaction is between the frames and the structural partitions. Considerable interaction between the frames and floor system also exists and is being studied in more detail in a related investigation.[21, 48, 49] An exact analytical treatment of frame-partition interaction is presented and a simplified approximate method of analysis is suggested, together with design examples. The results of an experimental investigation of a large size test building are also presented and correlated with the theoretical analysis.

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1. INTRODUCTION

1.1 Problem Statement

During recent years interest has developed in the actual structural behavior of buildings. Several studies have been conducted on various types of structures. Results of completed work and observations made to date give every indication that calculated stresses, deflections and maximum carrying capacities of steel frames using present methods of analysis do not correspond to their actual values. This of course is due to the fact that buildings are not just two dimensional plane frames. In this regard it is appropriate to quote a recent statement made by Professor G. Winter of Cornell University. Reporting on tall buildings to the American Regional Conference at Chicago he said, "I myself have never seen a two-dimensional building".(46) Completed buildings have other elements such as floors, roof, exterior curtain walls and interior partitions, all of which stiffen the frames and combine forces with them to resist the applied loads. The interaction between all the elements present in a building should be considered if a more rational and economical design is to be achieved.

Among the structures which are now widely used and which will probably attain even greater use in the near future are the modern apartment or office multistory buildings. One of the major problems which faces the designers of these buildings is to limit their drift under wind or seismic loads. Drift limitation at working wind loads is required chiefly to maintain the comfort of the occupants, ensure the proper functioning of the various services within the building and to avoid damage to non-structural elements such as exterior curtain walls and interior partitions. Drift can be controlled efficiently and economically by provision of a suitable bracing system. However, due to architectural consideration this is not always possible and an unbraced frame design has to be adopted.

Multistory unbraced frames of more than about twenty stories and designed for strength only often need to be redesigned with substantial increase in steel weight (mainly in the beams) to bring building drift down within reasonable limits. This additional weight in steel and more can be saved by taking into account the stiffening effect of other elements present in the building.

1.2 Purpose of Investigation

The main purpose of the present investigation is to study both theoretically and experimentally the structural interaction between the frames, the floor and wall systems in unbraced multistory frames and develop a method whereby the stiffening effects of these elements can be included in design.

The floor system is assumed to be either solid concrete or concrete on metal deck both of which are made composite with the frame beams. The wall system refers to exterior curtain walls and interior partitions. For the purpose of this report these walls and partitions will be referred to as structural partitions to differentiate them from the more rigid shear walls which are provided in some multi-story buildings to resist lateral loads. Although structural partitions may be made of any shear resisting elements which are adequately connected to the frames, emphasis in this report will be made on partitions made of light-gage steel sheeting.

The work at Lehigh University on the frame-floor interaction is still underway. On completion it will be reported in detail elsewhere.[21] Apart from a brief account of the frame-floor interaction, this report is focused on the stiffening effect of structural partitions.

1.3 Frame-Floor Interaction

It is common practice nowadays to make the concrete floor composite with the frame beams. As a result, the bending rigidity of the beams in the positive moment region is usually more than doubled. Current composite design procedure allows for this effect only for the gravity loading condition. Composite beams can also affect the frame behavior under the combined loading condition. Under this loading condition, the composite beams are subjected to concentrated moments at the beam-column joints. Since the concrete floor surrounds the columns, compression is developed between the leeward face of each column and the adjacent concrete. This makes it possible to utilize the composite action in the positive moment regions that develop in the beams near their windward ends. The relative lengths of the positive moment regions depend on the relative values of the gravity and lateral load moments. Preliminary results of the work currently conducted at Lehigh University indicate that this type of frame-floor interaction leads to a significant reduction in building drift. In a recent paper on the analysis of composite structures by Babb, the author found that the mere presence of floor slabs in his test building reduced the drift of the frames under lateral loads to only 60% of the bare frame value.(6) When the two end frames in the building were braced the floor slabs acting as diaphragms further reduced the drift of all the frames to the extent that measured deflections and stresses became virtually negligible.

1.4 Frame-Wall Interaction

Exterior curtain walls and interior partitions are conventionally used for the sole purpose of excluding the weather or subdividing the floor area into separate units. In multi-story buildings these partitions can be designed to interact with the frames to carry the lateral loads with appreciable reduction in building drift and large savings in the steel members of the frames. Structural partitions can affect the overall behavior of the building in various ways. Their most significant effect, however, is the resistance they offer to building drift-by virtue of their shear stiffness. This effect will be considered at length in this report.

1.5 Requirements of Partitions

A structural partition is, by name, a structural element and as such it should first of all be permanent. Figure 1 shows a floor plan and a typical cross frame in a steel building where structural partitions can drastically reduce building drift.(1) Indicated on the floor plan are partitions which may be considered structural as well as other removeable non-structural partitions.

A structural partition should possess adequate shear strength and stiffness. A partition made of light-gage steel sheets answers all these requirements. Plain sheets, however, are not suitable as they have very small out of plane bending stiffness while partitions should be stiff enough to resist incidental lateral loads and to facilitate handling during erection. Also they are likely to buckle and reach a state of pure tension field at very low shear loads. Having flexible edge members, as anticipated, such panels exhibit a highly nonlinear behavior associated with a drastic reduction in their shear stiffness.(9) Corrugated (or ribbed) steel sheets have none of these drawbacks while possessing other desirable qualities, such as durability, low cost and light weight. Further, architectural requirements such as attractive surface finish, sound proofing and heat insulation can be easily provided in combination with these sheets at relatively small extra cost.

For these reasons corrugated steel sheets are favored as a suitable material for structural partitions.

1.6 Frame-Partition Connections

Connections between partitions and frames play a very important role in their integrated behavior. There are several possible details for such connections. In order to decide on the most suitable detail the following criteria were established.

1. The connections should be capable to transfer shear load from the frame to the partition and allow the sheet to be in pure shear; a state of stress to which it is particularly suited.

2. Connections should be detailed so as the partition may not interfere with the flexure of the bounding beams and columns. This is desirable first to relieve the rather slender sheeting from any compressive forces resulting from bending of the bounding members, and second to maintain the frame action and facilitate the clad frame analysis.
3. Connection details should be as simple as possible if they are to become a practical proposition.

Connection details to meet all these requirements consist of edge members fastened to the sheets along the four edges and then the whole panel is connected to the frame at the four corners. These connection details will be discussed further in Section 2.3.

1.7 Previous Work

The previous work which has bearing on the investigation presented in this report lies in two research areas concerning the structural application of light gage corrugated metal sheets and the behavior of shear panels made of these sheets. These will be reviewed separately below.

1.7.1 Structural Applications of Light Gage Corrugated Steel Diaphragms

Diaphragms made of light gage corrugated sheets have been widely used in building construction due to their inherent high shear strength and stiffness. The first conscious use of these diaphragms in design was to rely on them to provide lateral support to floor beams and roof purlins. This type of restraining effect was studied both theoretically and experimentally by Errera, Pincus and Fisher.[24] However this is a local effect which does not affect the overall behavior of buildings. Diaphragms were next relied on to resist lateral loads from wind or earthquake forces and bridge these forces over to a plane of vertical bracing which could be a suitable bracing system, shear wall or other light gage diaphragms.

The first organized work on the shear behavior of light gage diaphragms and their structural applications was carried out at Cornell University. In 1960, Nilson introduced the simple concept of diaphragm action and showed how the roof sheeting can perform the function of conventional cross-bracing systems.[39] This and other work done at Cornell form the basis of a manual produced by the American Iron and Steel Institute for the design of light gage diaphragms.[2] Nilson extended the application of diaphragm action to steel roof shells in which the surfaces are mainly in shear. These included folded plate construction and hyperbolic paraboloids.[40,41] Both were tested and their performance was found to be satisfactory. Since then many such roofs have been constructed in the USA and abroad.

In 1960 a research program to investigate the behavior of sheeted single story industrial buildings started at Manchester University.[22] Bryan and El-Dakhakhni presented a basic theoretical treatment of clad frames which is applicable for any loading conditions.[10] By separating the sway (or spread) moments from the non-sway (or non-spread) moments the basic treatment was developed into a very simple method whereby the stiffening effect of roof cladding could be easily allowed for and utilized in design. The theory pointed out the prime importance of the stiffness of the frame relative to the shear stiffness of a roof panel. In fact it was shown that the stiffening effect of roof cladding depends solely on this relative stiffness factor. The developed theory was verified beyond any doubt by an extensive testing program which included tests in the elastic as well as the plastic ranges on models, a semi-full size building and field tests.[11,7,12,17] This research work culminated in the production of a manual for stressed skin design.[8]

Recently the effect of roof cladding on the elastic stability of portal frames was investigated by El-Dakhakhni and Daniels.[23] The buckling load of frame columns was expressed in terms of two non-dimensional factors representing the flexural restraint offered by the frame beam and the translational restraint offered by the roof sheeting. The investigation demonstrated the appreciable effect the roof cladding has on the buckling strength of frames. It also presented design curves for the effective column length factor allowing for the roof effect.

The most recent application of light-gage diaphragms is its use as partitions in multi-story buildings. While performing the normal function of dividing the floor plan into separate service areas, with proper detailing these partitions can be used economically and efficiently in controlling building drift under working lateral loads.

A great deal of work has been done on the problems of frame-shear-wall interaction. Perhaps the first attempt in this research area is that made by Khan and Sbaraunis.[31] This attempt was followed by others.[36]

A great deal of work has also been done on the testing of single-story frames infilled with masonry or concrete walls and the analysis of multi-story frames with such infill walls. The research work of Malcolm Holmes and Stafford Smith is the best known in this area.[26,44,45]

On the other hand, relatively few efforts have been made to study the interaction between multi-story frames and light gage partitions. Further, in the few available literature no experimental study has been made or even reported to have been done elsewhere.

Among the three known investigations on clad multi-story frames, one only has been reported.[20] The information about the other two was obtained through direct contact with the investigators.[42,37] The three investigations will be reviewed separately below:

1. Work at Karlsruhe University - Germany

At the International Conference on Tall Buildings held at Lehigh University in August 1972, Dubas[20] presented a paper on the interaction of structural elements with cladding. He pointed out the possibility of using light gage corrugated steel sheets as vertical diaphragms and a number of factors that need special attention such as fastenening details, openings, and the effect of repeated loading.

He described the design of a building composed of frames coupled with partitions and other shear resisting elements, and discussed the results obtained by continuous and discrete methods of analysis. This paper is based on unpublished investigation by Rubin.[43]

2. Work at Cornell University - USA

The investigation at Cornell University is divided into two separate parts. The first part is mainly concerned with the analysis of clad multi-story frames for assumed partition shear characteristics. The second part deals with the shear stiffness of light gage partitions whose determination is essential to the analysis of clad frames. The investigation thus covers the same research area considered in this report. The first part of the investigation conducted by Miller[37] is reviewed here, while the second part by Ammar[3] will be discussed in Sect. 2.3.

Miller developed a linear elastic analysis computer program which is capable of analyzing clad multi-story frames. It requires over 300^k reserved locations in the central memory of the computer and all calculations are done in double precision. The program utilizes a routine developed by Irons[29] for the efficient solution of large number of linear simultaneous equations. In the proposed elastic analysis the partitions are assumed to be connected in such a manner that they are able to transfer shear without being subjected to axial forces caused by the gravity loads on the frame beams. However, the two connection details suggested by Miller, may allow some load transfer from the frame beam to the partitions.

This does not only cause the rather slender sheeting to buckle prematurely but would change the frame action on which the entire analysis is based. In order to study the behavior of clad frames Miller used analytical examples of three single bay single story frames made of light, medium and heavy member sizes in conjunction with partitions made of three sheet thicknesses. The chosen examples while useful in studying the shear behavior of partitions are not suitable to study the behavior of clad multi-story frames. This is because the load distribution in single bay, single story, clad frames depends solely on the lateral stiffness of the bare frame relative to the shear stiffness of the partition, which is not true in multi-story frames. Miller also investigated the influence of cladding on the behavior of a 26 story 3 bay frame and presented a method to calculate the buckling load of shear panels made of sheets having trapezoidal profile. He concluded that they possess adequate buckling strength to be useful in multi-story buildings.

3. Work at Cambridge University - England

Concurrently with the work described in this report and the work at Cornell University which is reviewed above, an investigation into the effect of cladding on tall buildings was conducted by Oppenheim at Cambridge University[42]. Oppenheim studied the problem at length and suggested three design stages. He then proceeded with the analysis of the first stage where the partitions are called upon to control drift only. A sparse matrix technique was used to reduce the storage and computation time and an iterative method was chosen so that it might be used to study the response of clad frames to dynamic loading which the investigator also made. An approximate method based on a single story stiffness and partitions with varying stiffness along the height of the building was suggested and its results were checked against those obtained from the developed exact method. To demonstrate the effect of cladding the investigator considered four sample frames designed by Morino.[38] These structures were chosen as they demonstrated the typical problem encountered in practice of frames possessing adequate strength but having excessive drift under lateral working loads. All four frames were subjected to static analysis and one to dynamic analysis. The results indicated that excessive building drift could be controlled efficiently by utilizing the stiffening effect of the partitions. A significant part of the Cambridge University investigation was devoted to the analysis of the shear stiffness of plane isotropic panels. Distinction was made between what was termed a pure shear panel and a corner connected panel. The shear stiffness of the latter was obtained from a finite element analysis. According to the connection details suggested by Oppenheim, corner-connected partitions are impracticable as they would need complicated connection details to distribute the concentrated loads developed at the corners to the skin. Also, the stress concentrations at the corners would result in an appreciable reduction in the partition strength and stiffness.

It should be pointed out here that this is not always the case with all corner-connection partitions. In fact, it will be shown in this report that the partition can be corner-connected while the skin remains in a state of pure shear.

1.7.2 Shear Behavior of Corrugated Shear Panels

A considerable amount of work on shear of corrugated sheet panels has been done in the past twelve years. Nilson laid the foundation of the work by carrying out almost 40 tests on large scale diaphragms.[39] He noted the warping of the corrugation profiles that takes place at the ends of the panels and suggested end closures to eliminate its effect. He also noted the effect edge and seam fasteners have on panel strength and stiffness. No attempt was made to develop any theoretical or empirical formula for the strength and stiffness of panels but it was concluded that they decrease as the panel span increases.

Luttrell extended Nilson's work by carrying out tests on another 60 large scale diaphragms.[35] He investigated the effect of panel configuration, material properties, span length and paid particular attention to the method of fastening the diaphragm. He also developed a semi-empirical formula for estimating the shear stiffness of corrugated panels. This formula allows for only the shear deformation of the sheet and the effect of end warping which, according to the investigator, was constant for a certain panel regardless of its depth. Accordingly the end warping effect could be found by testing a relatively shallow diaphragm and using the results for similar diaphragms of any depth. The work of Luttrell and the earlier work of Nilson form the basis of the design recommendations in Ref. 2.

Apparao carried out further tests and concluded that the shear stiffness of a panel was mainly dependent on its length and the type and spacing of fasteners, and its strength depended on the thickness of the sheet and the type and spacing of fasteners[4]. He also suggested an empirical relation for panel strength.

The early work at Manchester University has shown that the effect of cladding can be easily allowed for inelastic as well as plastic design of industrial buildings provided that the diaphragm action of a roof panel can be predicted[20,10,11,7,12]. In the mean-time it was apparent that there was no suitable general theory for determining diaphragm stiffness and strength. This state was expressed in a paper by Errera, Pincus and Fisher published in February 1967 where they stated, "To the writer's knowledge no suitable general theory for determining diaphragm rigidity is available"[24,25].

The first attempt to fill that gap was made by Bryan and Jackson[16]. Although that investigation was of a preliminary nature, it served its purpose of pointing out the major factors influencing the shear behavior of panels.

The first and so far the only general theory for predicting diaphragm behavior was developed by Bryan and El-Dakhakhni[13]. Analytical expressions were derived for the various factors affecting the stiffness of shear diaphragms. These factors included the deformation of purlin-rafter connections, slip at sheet-purlin fasteners and sheet-sheet or seam fasteners, shear deformation of the sheet, axial deformation of the edge members, and bending and twisting of the corrugation profiles. The effect of intermittent fastening of the corrugation and the stiffening effect of intermediate members were also included. The theory derived was verified by extensive testing on component parts as well as large scale panels.[14,15] Nearly 150 tests are reported in Ref. 14 alone. In all these tests it was noticed that the strength of the fasteners controlled the strength of the panels. In the tests on lighter panels reported in Ref. 15 failure occurred due to instability of the edge member.

Further theoretical investigation was conducted at Manchester University by Horne and Raslan.[27] Energy and finite difference approaches were considered and the effect of curving of the corrugation generators was included.

Very recently Libove and associates carried out a theoretical study with the purpose of determining the shear stiffness, the deformations and the stresses in panels made of trapezoidally corrugated plates with discontinuous attachment at the ends.[33,34,28] The theory employs the principle of minimum potential energy and as done by Horne and Raslan it takes into account the effect of curving of the corrugation generators. A comprehensive bibliography of relevant work carried out mainly in connection with aircraft structures is presented in a paper by Libove entitled, "Survey of Recent Work of the Analysis of Discretely Attached Corrugated Shear Webs".[32]

With the exception of the work done by Libove and his associates, which was developed in connection with aircraft structures, all the investigations reviewed above were conducted in relation to roof or deck diaphragms.

1.8 Plan of Treatment

In addition to Chapter 1, which presents an introduction to the investigation, this report contains five more chapters. Chapter 2 includes the theoretical analysis. Chapter 3 describes a large scale test building. Experiments carried out on the test building in the elastic range are presented in Chapter 4. Maximum strength experiments are presented in Chapter 5. Chapter 6 is devoted to the general conclusions and recommendations.

Chapter 2 is subdivided into three sections. Section 2.1 includes a basic theoretical treatment of clad multi-story frames. The method suggested is based on computing the flexibility coefficients of the bare frame and is applicable regardless of the material used for the partitions. The stiffening effect of composite floors can be readily included by replacing the influence coefficients of the bare frame by those of the frame having composite beams. Section 2.1 also includes an approximate method for estimating a clad frame drift associated with a given partition. The validity of this method is checked against the results obtained from the exact analysis of the design example presented in Sect. 2.3. The approximate method is found to give a good estimate of the shear flexibility of partitions that would be required to reduce frame drift to a prespecified smaller value. In Sect. 2.2 the shear flexibility of corner connected partitions is discussed. It is divided into a number of components representing the contributions of the various factors involved. Following similar procedure to that adopted previously by Bryan and El-Dakhakhni and using energy methods, separate expressions are derived for individual components and the final result is obtained by adding up the relevant values. The method is attractively simple. Only short hand calculations are necessary to determine the shear flexibility of a partition. Also, by separating the various effects the method has the additional advantage of pointing out the relative importance of the various factors and thus helping the designer to make easy and quick adjustments to the partition flexibility according to his needs. A design example of a clad frame is presented in Sect. 2.3. The frame has two bays and fifteen stories. Although it is realized that the drift of this frame under working loads is not excessive and lies within the acceptable limits yet it is introduced to demonstrate the effect of partitions of different flexibilities and to check the validity of the approximate method.

The basic theoretical treatment and the design example presented in Chapter 2 should provide ample understanding of the elastic behavior of clad multi-story frames.

Chapters 3, 4 and 5 include the experimental investigation. Chapter 3 presents some aspects of planning, design, loading arrangement and instrumentation of a large size test building constructed in the laboratory to study the various structural interactions in multi-story buildings. As planned and designed this test building provides the capability of studying the integrated behavior of multi-story buildings under gravity, lateral and combined loading conditions. However, due to unforeseen reasons tests were conducted for the lateral loading condition only. Included in Chapter 4 are a description and a discussion of tests carried out on the test building in the elastic range. These consist of tests on the bare frames, frames with composite floors, and frames with composite floors and structural partitions. Throughout the experimental work emphasis was placed on the effect of partitions. Two partitions made of corrugated steel sheeting and with aspect ratio of about 1 and 2 were tested. Each partition was tested with different fastening details between the sheeting and the edge members. The results of those tests are discussed in the light of the theory presented in Chapter 2 and good agreement is shown to exist between calculated and observed values.

Near the end of the research program an opportunity arose to conduct failure tests. To the investigator's knowledge this is the first time that a large size test building with partitions made of light gage sheeting has been loaded to its maximum capacity. The results of these tests are reported in Chapter 5.

General conclusions and recommendations are presented in Chapter 6.

2. THEORETICAL ANALYSIS

2.1 General

At the outset, it was felt that the best way to treat the practical problem of the integrated behavior of multi-story buildings is to follow an engineering approach. To engineers simplicity of the method is highly desirable because the value of even the most sophisticated method decreases sharply if it requires long computation time or exceptionally large computers. Flexibility of the method is also desirable in that it should allow adjustments, which are inevitable in design, to be made at minimum additional effort and cost. After all it goes without saying that if a problem can be solved in more ways than one, it pays to use the quickest and simplest so long as the accuracy of the solution is not sacrificed.

The method suggested here for the analysis of clad frames has the characteristic of being both simple and adjustable. It is based on breaking up the original problem into two smaller problems. The first concerns the behavior of the bare frame alone, while the second deals with the shear flexibility of structural partitions.

In the first problem it is only necessary to compute the flexibility coefficients of the bare frame. This is done within a relatively short time compared to that for the analysis of the frame under working loads. This is because in general the analysis of a frame having m joints calls for the solution of a set of 3m linear simultaneous equations of the form:

$$\{F\} = [K] \{D\} \quad (1)$$

where $\{F\}_{3m \times 1}$ is the load vector, $[K]_{3m \times 3m}$ is the structure stiffness matrix and $\{D\}_{3m \times 1}$ is the joint displacement vector. A solution for the displacements is obtained by inversion of the stiffness matrix K. Thus,

$$\{D\} = [K^{-1}] \{F\} \quad (2)$$

Equation 2 can be generalized to include any number of loading conditions and will then assume the following form:

$$\{D\} = [K^{-1}] [F] \quad (3)$$

where $[F]$ in this case is a 3mxn matrix; n being the number of loading conditions which in our case will be equal to the number of stories.

Among all the operations involved, inversion of the stiffness matrix K is the lengthiest and most expensive. Once K^{-1} is generated, the flexibility coefficients are obtained by the fast operation of post multiplying it by individual columns of F where all the elements in each column but one are zero.

It is realized that matrix inversion is not the most efficient method for solving the large number of equations involved but a method will be suggested in subsection 2.2.3 to reduce the number of unknowns.

In the second problem, the same philosophy of tackling a complex problem by breaking it into a number of smaller problems was again adopted. The shear flexibility of a partition is subdivided into a number of components representing the contributions of the various factors involved. Separate expressions are derived for these components and only short hand calculations are necessary to find their values. The final result is then obtained by adding up the values of the relevant components. By separating the various effects, the method has the advantage of pointing out the relative importance of each factor, and helping the designer to make easy and quick adjustments to the partition flexibility according to his needs.

2.2 Stiffening Effect of Structural Partitions

2.2.1 Basic Analytical Treatment

Consider the two-bay multi-story frame shown in Fig. 2 and assume it has partitions in the right bay only. Under a set of lateral loads H_i the frame will drift through a set of horizontal displacements Δ_i at various floor levels. A set of restraining forces H_i' will be developed by the partitions as they are forced through some shear displacements preventing further drift.

For the frame and using the common notation for the flexibility coefficients δ_{ij} the actual displacements at various floor levels will be given by:

$$\begin{aligned}
\Delta_1 &= H_1\delta_{11} + H_2\delta_{12} + \text{-----} + H_n\delta_{1n} - (H_1'\delta_{11} + H_2'\delta_{12} + \text{-----} + H_n'\delta_{1n}) \\
\Delta_2 &= H_1\delta_{21} + H_2\delta_{22} + \text{-----} + H_n\delta_{2n} - (H_1'\delta_{21} + H_2'\delta_{22} + \text{-----} + H_n'\delta_{2n}) \\
\Delta_3 &= H_1\delta_{31} + H_3\delta_{32} + \text{-----} + H_n\delta_{3n} - (H_1'\delta_{31} + H_2'\delta_{32} + \text{-----} + H_n'\delta_{3n}) \qquad (4) \\
\Delta_n &= H_1\delta_{n1} + H_2\delta_{n2} + \text{-----} + H_n\delta_{nn} - (H_1'\delta_{n1} + H_2'\delta_{n2} + \text{-----} + H_n'\delta_{nn})
\end{aligned}$$

For the partitions and denoting the shear flexibility of a panel by C the actual displacements at various floor levels will be given by:

$$\Delta_1 = C\,[n\,H_1' + (n-1)\,H_2' + (n-2)\,H_3' + \text{---------} + H_n']$$

$$\Delta_2 = C\,[(n-1)\,H_1' + (n-1)\,H_2' + (n-2)\,H_3' + \text{----} + H_n']$$

$$\Delta_3 = C\,[(n-2)\,H_1' + (n-2)\,H_2' + (n-2)\,H_3' + \text{----} + H_n'] \qquad (5)$$

$$\Delta_n = C \sum_1^n H'$$

By equating the frame deflections in Eqs. 4 to the deflections of the partitions in Eqs. 5 the following set of compatibility equations is obtained

$$H_1'[\delta_{11} + nC] + H_2'[\delta_{12} + (n-1)C] + H_3'[\delta_{13} + (n-2)C] + \text{----} + H_n'[\delta_{1n} + C] = \Delta_{1b}$$

$$H_1'[\delta_{21} + (n-1)C] + H_2'[\delta_{22} + (n-1)C] + H_3'[\delta_{23} + (n-2)C] + \text{---} + H_n'[\delta_{2n} + C] = \Delta_{2b}$$

$$H_1'[\delta_{31} + (n-2)C] + H_2'[\delta_{32} + (n-2)C] + H_3'[\delta_{33} + (n-2)C] + \text{--} + H_n'[\delta_{3n} + C] = \Delta_{3b}$$

$$H_1'[\delta_{n1} + C] + H_2'[\delta_{n2} + C] + H_3'[\delta_{n3} + C] + \text{-----------} + H_n'[\delta_{nn} + C] = \Delta_{nb} \qquad (6)$$

The right hand sides of Eqs. 6, Δ_{1b}, Δ_{2b},---Δ_{nb} are the drift at various floor levels of the bare frame due to the applied lateral loads H_1, H_2,---H_n and are given by:

$$\Delta_{ib} = H_1\delta_{i1} + H_2\delta_{i2} + H_3\delta_{i3} + \text{------------} + H_n\delta_{in} \qquad (7)$$

In matrix shorthand, Eqs. 6 may be expressed as:

$$A\,H' = B \qquad (8)$$

where A is a square symmetrical matrix which can be calculated by a simple routine using the flexibility coefficients, the number of stories n and the shear flexibility of a partition C.

B is a column vector representing the displacements of various floor levels due to the applied loads and may be obtained from Eq. 7.

By solving Eq. 8 the restraining forces H' are obtained. The actual frame drift is obtained by back substitution in Eqs. 4. The shear forces on the partitions can be obtained from Eqs. 5.

Assuming that the flexibility coefficients are generated by any of the many programs available for linear elastic analysis and stored in matrix D(N,N) where in FORTRAN, N refers to the number of stories, the program segment to generate the A matrix and B vector in Eq. 8, and produce the restraining forces, the net forces on the frame, HNET, the shearing forces in the partitions, SF, and the actual displacements, DACT is as follows:

```
C       CALCULATION OF MATRIX A
        DO 3 J = 1,N
        IF (I.GT.J) GO TO 4
        A(I,J) = D(I,J) + C* (N + 1 - J)
        GO TO 3
4       A(I,J) = D(I,J) + C* (N + 1 - J)
3       CONTINUE
C       CALCULATION OF VECTOR B
        DO 5 I = 1, N
        SUM = 0.0
        DO 6 J = 1, N
6       SUM = SUM + D(I,J)
        B(I) = H*SUM
5       CONTINUE
C       SOLVE FOR THE RESTRAINING FORCES AND STORE THEM IN B
        CALL SOLVE (A,B,N.L, DET)
        CALL OUTE (B,N,L,9HRESTRAINT, 7H FORCES)
```

```
C       CALCULATE NET FORCES ON FRAME AND S.F. IN PARTITIONS
        WRITE (6,7)
7       FORMAT (1H, * NET FORCES ON FRAME   S.F. IN WALL *)
        Q = 0.0
        DO 8 I = 1,N
        HNET (I) = H - B(I)
        Q = Q + B (I)
        SF(I) = Q
8       WRITE (6,9) HNET(I), SF(I)
9       FORMAT (3X, E14.7,8X,E14.7)
C       CALCULATION OF ACTUAL DISPLACEMENTS
        WRITE (6,13)
13      FORMAT (1H, 20HACTUAL DISPLACEMENTS)
        DO 10 I = 1,N
        DACT = 0.0
        DO 11 J = 1,N
11      DACT = DACT + HNET (J)*D(I,J)
10      WRITE (6,12) DACT
12      FORMAT (1H ,E14.7)
        STOP
```

SOLVE and OUTE are two subroutines from FLMXPK matrix package used to solve the equations and output of the results.[30]

2.2.2 Approximate Method for Estimating Drift of Clad Frames

For design purposes, it is important to have a simple method to estimate even approximately the drift of a clad frame associated with a specific shear flexibility of partitions or, more to the point, to estimate the shear flexibility of a partition that would be required to reduce frame drift by a certain amount. The approximate method is derived from the basic analytical treatment given in subsection 2.2.1 by making the following simplifying assumptions.

1. The frame beams have infinite flexural stiffness. The bare frame will then drift in a racking mode and its resistance to lateral loads will depend on the stiffness of the columns only.

2. The frame columns have uniform sections along the height of the building.

It is not suggested that either of these assumptions is valid in practice but it is possible to select column sizes which, with these assumptions, will result in a drift comparable to that of the bare frame. Also, these assumptions are introduced to point out the prime importance of the stiffness of the bare frame relative to the stiffness of the partitions on the behavior of clad frames.

The shear flexibility C_c of the columns in a story is given by:

$$C_c = \sum \frac{L^3}{12\ E\ I_c} \tag{9}$$

where L = story height, I_c = moment of inertia of the column section about the axis of bending; the summation being carried out for all the columns in a story and E = modulus of elasticity.

Denoting the actual relative displacement between two consecutive floor levels, i and i + 1, by $\Delta_{i,i+1}$, then

for the bare frame $$\Delta_{i,i+1} = C_c \left(\sum_1^i H_i - \sum_1^i H_i'\right) \tag{10}$$

and for the partition $$\Delta_{i,i+1} = C \sum_1^i H_i' \tag{11}$$

Equating Eqs. 10 and 11,

$$\sum_1^i H_1' = \frac{C_c}{C_c + C} \sum_1^i H_i \tag{12}$$

From Eqs. 10 and 12

$$\Delta_{i,i+1} = C_c \sum_{1}^{i} H_1 \times \frac{C}{C + C_c} \qquad (13)$$

Since similar partitions are used in all stories, C is constant. Also, according to assumption 2, C_c is constant. Thus,

$$\Delta_{i,i+1} = \frac{C_c \; C}{C_c + C} \sum_{1}^{i} H_i \qquad (14)$$

The corresponding relative displacement in the bare frame is given by:

$$\Delta_{i,i+1} = C_c \sum_{1}^{i} H_i \qquad (15)$$

From Eqs. 14 and 15 and dropping similar displacement subscripts, the actual displacement Δ_{act} may be expressed in terms of the bare frame displacement Δ_{bare} as follows:

$$\Delta_{act} = \frac{1}{r + 1} \Delta_{bare} \qquad (16)$$

where r = relative flexibility factor given by:

$$r = \frac{C_c}{C} \qquad (17)$$

Equation 16 shows that $1/(r+1)$ represents a reduction factor to frame drift at any level. Equation 17 shows that this reduction factor depends solely on the relative partition-bare frame stiffness and that only when the partitions are very flexible relative to the frame will $r \rightarrow 0$ and hence $\Delta_{act} = \Delta_{bare}$.

The question that arises now is the column sizes in which story should be used in Eq. 9 to evaluate C_c. Since assumption 1 underestimates frame drift it will be reasonable to compensate this effect to use columns sizes nearer to the top of the frame. Of course a truly representative column size can not be decided upon until a sufficient number of multi-story frames is analysed and the resulting drift is compared to that calculated on the assumptions of rigid beams and uniform columns.

2.2.3 Simplifying Numerical Technique

Provided the flexibility coefficients are known, the time and cost for computing the restraining forces, net forces on the frame, the shearing forces on the partitions and the actual drift of the clad frame is relatively small. For instance, in the example presented in this report it took less than one second with execution cost of less than \$1.0 to produce all these values for the 15 story frame. The problem then lies in the computation of the flexibility coefficients and it is there that savings in the computation time need to be made.

In Eq. 3 we are interested in frame drift at various floor levels rather than the horizontal displacements of individual joints at each level. Since the axial deformations of the frame beams are usually very small and for all practical purposes can be neglected, it will be reasonable to assume the beams have infinite axial rigidity. While still allowing all the joints to displace vertically and rotate, this results in reducing the number of unknown horizontal displacements to one per story and hence the total number of unknowns from 3m to (2m + n). For instance in a 30-story 3-bay frame the number of unknown displacements will be reduced from 360 to 270. Clough, King and Wilson presented a method of analysis based on this assumption and suggested two methods for solving the resulting equations.[18] The first is an iterative scheme and the second is a recursive technique based on the tridiagonal nature of the stiffness matrix. Of the two the latter is more convenient to apply in conjunction with the method of analysis presented in this report.

The only new factor that appears in the preceeding section and needs further consideration is the shear flexibility of the partition C. Analytical expressions for this factor will be derived in Sect. 2.3.

2.3 Light Gage Structural Partitions

As explained earlier in the introduction, corrugated steel sheets are particularly suitable for structural partitions. The most important property of a partition is its shear flexibility. This property is governed by practical factors such as the method of fastening the sheets to the edge members or fastening individual sheet widths together, as well as theoretical consideration of the deformations of the component elements of the partition.

Considerable work has already been done on the behavior of corrugated shear panels.[2-4,13-15,27,28,32-35,39] However, with one exception, no work has been developed in relation with structural partitions. The exception made refers to the work currently underway at Cornell

University to predict analytically the shear flexibility of light gage partitions.[3] In that investigation a finite element approach is used together with complementary tests to determine a number of panel characteristics such as the stiffness of the fasteners, the shear stiffness of the corrugated sheet and the elastic modulus of the sheet in the weak direction. In deriving the partition stiffness, a rather fine mesh is required if a reasonable degree of accuracy is to be obtained. As reported, this would often cost more than the analysis of the frame.[37] The investigators at Cornell University are to be complimented for taking the challenge to solve the complex problem of the shear flexibility of partitions. However for such a problem influenced more by practical considerations of fastener details and less by theoretical considerations of deformations the long and costly computations are not entirely justified. This is particularly true when the solution will be used in conjunction with a multistory frame analysis where the size of the problem is already big and attempts are being made to cut it down.

Most of the previous work was developed in relation to roof and deck panels. The main differences betwen panels used for decks or roofs and those used for partitions are:

1. The shear flexibility of a roof or deck panel was defined as the displacement per unit shear load applied parallel to the corrugation generators. In the case of partitions the bay width is usually greater than the story height, and since higher out-of-plane stiffness is desirable it will be advantageous to install the panels

with the corrugation generators running vertically across the shorter span. Accordingly, the shear flexibility of a partition is defined here as the displacement per unit shear load applied normal to the corrugation generators.

2. Roof or deck panels can be fastened to the supporting purlins or floor beams on one face only. This leads to a discontinuity in the shear flow along the edges which results in bending and twisting of the corrugation profiles. These deformations in turn, have a profound effect on the panel stiffness. In the case of partitions the sheets can be fastened to the edge members on both faces of the corrugations. Bending and twisting can then be completely eliminated. Even if it is decided to fasten the sheets on one face only, the shear flexibility component due to bending has to be modified on account of the new measure of shear flexibility used for partitions. The investigation of this flexibility component is very important because partitions welded on both faces may prove so stiff that they may attract a share of the applied load in excess of their carrying capacity. In such cases, additional flexibility can be injected into the partitions by welding it on one face only allowing the free bending of corrugation profiles.

3. The shear flexibility of a roof or deck panel was derived assuming two corner connections. Connecting the partitions

at the four corners, as proposed here, leads to a new force distribution in the edge members which in turn affects the shear flexibility component due to axial deformation of the edge members. Other flexibility components are also affected as will be shown subsequently.

2.3.1 Shear Flexibitity of Corner-Connected Partitions

Figure 3 shows a typical structural partition. It consists of light gage corrugated sheets attached to edge members on the four sides. The sheets are installed with the corrugation generators vertical. The four edge members are assumed to be pin-connected to each other such that they have zero resistance to any applied shear. Individual sheet widths are fastened together along the seams and the whole sheet is fastened to the edge members along the perimeter. The assembled partition is connected to the steel frame only at the four corners. Under an applied shear load Q this partition will be subjected to the corner forces indicated in Fig. 4a.

As done before in regard to roof panels[(13)], various shear flexibility components of a structural partition are found separately and the total flexiblity is obtained by combining the relevant components. The main components are due to shear deformation of the sheet, axial deformation of the edge members, bending of the corrugation profile and local deformations (crimping) at the edge fasteners and seam fasteners. Thus,

$$C = C_1 + C_2 + C_3 + C_4 + C_5 \qquad (18)$$

The flexibility component C_1 - C_5 are derived subsequently.

2.3.2 Flexibility due to Sheet Deformation, C_1

Consider again the partition shown in Fig. 4a. Due to the loads applied at the corners and provided the sheet is continuously fastened to the four edge members it will be subjected to a uniform shear flow of Q/b around the edges as indicated in Fig. 4b. Assuming that the edge members have infinite axial rigidity and that the sheet is attached to them such that relative movement between both is prevented and that no bending of the corrugation profiles takes place then the shear displacement Δ will be only due to the shear deformation in the sheet.

$$u = \frac{q^2}{2G} \qquad (19)$$

where u = shear strain energy per unit volume, G = the shear modulus and q = shear stress which for a sheet thickness t is given by:

$$q = \frac{Q}{bt} \qquad (20)$$

The total strain energy U is thus given by:

$$U = \frac{q^2}{2G} (a \times \alpha\, b \times t) \qquad (21)$$

where a = partition width, b = partition length and α(a factor depending on the corrugation profile) = ratio of the developed length of a corrugation to its projected length.

From Eqs. 20 and 21

$$U = \frac{\alpha\, a\, Q^2}{2\, G\, bt} \qquad (22)$$

Equating the total strain energy in Eq. 22 to the external work done 1/2 $Q\Delta$ leads to:

$$C_1 = \frac{\Delta}{Q} = \frac{\alpha\ a}{G\ b\ t} \qquad (23$$

Substituting for G in terms of the modulus of elasticity E and Poissons ratio ν, Eq. 20 may be expressed as:

$$C_1 = \frac{2\alpha\ a(1 + \nu)}{E\ b\ t} \qquad (24)$$

2.3.3 Flexibility Due to Axial Deformation of Edge Members C_2

Assuming the sheet has infinite shear rigidity and that it is fastened to the edge members such that any relative movement between both is prevented and no bending of the corrugation profiles takes place, the shear displacement Δ will be due only to the axial deformation of the edge members.

Considering the equilibrium of one of the horizontal edge members it will be seen that the member is subjected to a uniform force per unit length equal in magnitude and opposite in direction to the shear flow in the sheet Q/b in addition to two end forces each of Q/2 acting as indicated in Fig. 4b. The axial force in the member thus varies linearly from a maximum value of Q/2 at the ends to zero at the middle. Referring again to Fig. 4b, the axial force at a distance x from the middle is $\pm$ Q x/b. Thus,

$$U = 2\int_0^{b/2} \frac{1}{2EA_1} \times \left(\frac{Qx}{b}\right)^2 dx = \frac{Q^2 b}{24\ EA_1} \qquad (25)$$

The strain energy of a vertical edge member can be found in a similar manner by noticing that the axial force in the member varies linearly from a maximum value of Qa/2b at the ends to zero at the middle.

$$U = 2\int_{o}^{a/2} \frac{1}{2EA_2} \times \left(\frac{Qy}{b}\right)^2 dy = \frac{Q^2 a^3}{24\ EA_2 b^2} \quad (26)$$

where A_1 and A_2 = cross sectional areas of a horizontal and a vertical edge member respectively.

Equating the strain energy of the two pairs of edge members to the external work done, $1/2\ Q\ \Delta$ leads to:

$$C_2 = \frac{\Delta}{Q} = \frac{1}{6E}\left(\frac{b}{A_1} + \frac{a^3}{A_2 b^2}\right) \quad (27)$$

If $A_1 = A_2 = A$,

$$C_2 = \frac{(b^3 + a^3)}{6\ EA\ b^2} \quad (28)$$

2.3.4 Flexibility Due to Bending of Corrugation Profile, C_3

As mentioned before when the sheet is fastened to the edge members on one face only, bending and twisting of the corrugations occur. Assuming that the sheet has infinite shear rigidity and the edge members infinite axial rigidity and that the sheet is fastened to the edge members in such a manner as to eliminate any relative movement between both but not interfere with bending and torsion of the corrugation profile then the shear displacement will be due to the latter two effects only. As the effect of torsion is negligible compared to that of bending, partition flexibility due to bending only will be considered.

Figure 5 shows two identical shear panels with the corrugation generators vertical. Both panels are loaded similarly by the indicated set of shearing forces in equilibrium. If the vertical edge member a d is held in position the panel will deflect vertically on amount Δ_v as shown in Fig. 5a. If on the other hand the horizontal edge member a b is held in position the panel will deflect horizontally an amount Δ_h as shown in Fig. 5b. Since in both cases the shear distortion γ is the same the two displacements may be related. Thus,

$$\gamma = \frac{\Delta_v}{b} = \frac{\Delta_h}{a} \tag{29}$$

$$\Delta_h = \frac{a}{b} \Delta_v \tag{30}$$

Under a shear load Q applied parallel to the corrugation generators, Δ_v resulting from bending of the corrugation profile has been found in Ref. 13.

$$\Delta_v = n_c \times \frac{144 K h^3 \ell^2}{E t^3 a^3} Q \tag{31}$$

where h = height of corrugation, ℓ = width of crest of corrugation, n_c = number of corrugations in panel and K = factor depending on the geometry of the corrugation profile and how the sheet is attached to the edge member. This factor will be discussed further later.

From Eqs. 30 and 31

$$\Delta_h = n_c \times \frac{144 K h^3 \ell^2}{E t^3 a^2 b} Q \tag{32}$$

Equation 32 gives Δ_h due to a horizontal shearing force Qb/a. Due to a horizontal shearing force Q, Δ_h will thus be given by:

$$\Delta_h = n_c \times \frac{144 K h^3 \ell^2}{E t^3 a^2 b} Q \times \frac{a}{b} \tag{33}$$

Substituting for n_c by its value, $n_c = \frac{b}{d}$, where d = pitch of corrugations, Eq. 33 reduces to:

$$\Delta_h = \frac{144\ K\ h^3\ \ell^2}{E\ t^3\ a\ b\ d}\ Q \tag{34}$$

$$C_3 = \frac{\Delta_h}{Q} = \frac{144\ K\ h^3\ \ell^2}{E\ t^3\ a\ b\ d} \tag{35}$$

The factor K appearing in Eq. 35 is defined with reference to Fig. 6.

$$K = \frac{EI\ \delta}{h^3} \tag{36}$$

where EI = bending rigidity of the sheet per unit length along the corrugation generator, δ = crest deflection due to unit horizontal load.

If the sheet is welded to the edge member at the center of the valley as shown in Fig. 6a, then it can be easily shown that K is given by:

$$K = \frac{(d + 2h)(d^2 - 3\ell d + 3\ell^2)}{12\ h\ d^2} \tag{37}$$

If the sheet is continuously welded along the lower face or just welded at the toes of the corrugations as shown in Fig. 6b, K will be given by:

$$K = \frac{(2\ell + 3h)}{12(\ell + 6h)} \tag{38}$$

It will be appropriate to mention here that an excellent review of welding details of light gage steel diaphragms has been presented by A. Nilson in Ref. 39. A more recent study may be found in Ref. 19.

2.3.5 Flexibility Due to Crimping at Sheet-Edge Members Fasteners, C_4

In general the sheet will be fastened to the edge members by spot welds at points spaced along the perimeter. Shear transfer between the sheet and the edge members at these points results in local deformation of the light gage material. These local deformations around the fasteners, usually referred to as crimping, contribute to the partition flexibility. In order to evaluate this effect consider the same partition as before and let it be assumed that both the sheet and edge members have infinite rigidities, then under the applied shear load Q the displacement Δ will be due to sheet crimping only.

Let the fasteners along the horizontal edge members be spaced at p_b and those along the vertical edge members at p_a, and let the relative movement due to sheet crimping at each fastener be s per unit load.

$$\text{Force per fastener along horizontal member} = \frac{Q}{b}\, p_b \qquad (39)$$

$$\text{Movement at fastener} = s\left(\frac{Q\, p_b}{b}\right) \qquad (40)$$

Work done at all fasteners along horizontal members

$$= \frac{s}{2}\left(\frac{Q\, p_b}{b}\right)^2 \left(\frac{2b}{p_b}\right) = \frac{s\, p_b\, Q^2}{b} \qquad (41)$$

Similarly,

Work done at all fasteners along vertical members

$$= \frac{s\, p_a\, a\, Q^2}{b^2} \qquad (42)$$

Equating the work done at all fasteners to the external work done leads to:

$$C_4 = \frac{\Delta}{Q} = \frac{2s}{b^2} (p_b\ b + p_a\ a) \tag{43}$$

If $p_a = p_b = p$,

$$C_4 = \frac{2\ s\ p}{b^2} (a + b) \tag{44}$$

2.3.6 Flexibility Due to Crimping at Seam Fasteners , C_5

In deriving all the previous flexibility components it was assumed that the sheet was made up of one piece. In practice, although sheets are produced in lengths that exceed the requirement for normal story height they are available only in standard widths which vary from 2 ft. to 3 ft. If the partition is to behave, as assumed, as a shear panel individual sheets have to be fastened together along the seams. Crimping at the seam fasteners result in additional flexibility of the partition. In order to evaluate the effect consider the same panel as before and assume all its components have infinite rigidities then under the applied shear load Q the displacement Δ will be only due to crimping at the seam fasteners.

Let the fasteners along each seam joint be spaced at p' and the relative movement due to sheet crimping at each fastener be s' per unit load.

$$\text{Force per fastener} = \frac{Q}{b}\ p' \tag{45}$$

$$\text{Work done at one fastener} = \frac{s'}{2} \left(\frac{Q}{b}\ p'\right)^2 \tag{46}$$

$$\text{Work done at all seam fasteners} = \frac{s}{2}\left(\frac{Q}{b}\,p'\right)^2 \times \frac{a}{p'} \times \frac{b}{b_o}$$

$$= \frac{1}{2}\,\frac{Q^2 a\,s'p'}{b\,b_o} \qquad (47)$$

where b_o = average sheet width.

Equating the work done at all fasteners in Eq. 47 to the external work done leads to:

$$C_5 = \frac{\Delta}{Q} = \frac{a\,p's'}{b\,b_o} \qquad (48)$$

Factors s and s' appearing in Eqs. 44 and 48 respectively depend mainly on the sheet thickness and the size of the weld. They will have to be determined experimentally but this creates no real difficulty since all that is needed is a simple pull out test. An empirical formula based on similar tests carried out on self tapping screws is presented in Ref. 13. Further tests on welded connections may be found in Ref. 19.

2.4 Design Example

A two-bay fifteen-story frame was analysed taking the stiffening effect of structural partitions into account. The frame was designed according to 1969 AISC allowable stress provisions. The overall dimensions and member sizes are shown in Fig. 7. The frame was spaced at 30 ft. Thus, with wind intensity of 20 psf the equivalent wind loads will be 7.2 kip per joint as indicated. Three shear flexibility values for the partitions were used. These correspond to the properties quoted in the description of each partition type.

2.4.1 Description of Partition Type 1

The partition is assumed to be installed in one bay only. For a depth of the column section of 14 in. the panel length is:

$$b = 30 \times 12 - 14 = 346 \text{ in}$$

For an average frame beam depth of 30 in. the panel width is:

$$a = 12 \times 12 - 30 = 114 \text{ in.}$$

The sheet used is of 18 gage (thickness $t = 0.0478$ in.) and has a corrugation profile 3 in. wide and 1.5 in. deep. The edge member is a channel 2 in. deep and 0.25 in. thick so that the 1.5 in. deep sheet profile can fit within its flanges. The cross sectional area of the edge member $A = 0.875$ in.^{2} The sheet is assumed to be continuously welded to the edge members so that sheet crimping at the connections to the edge members is negligible, $s = 0$. Average width of individual sheets $b_o = 34$ in. and the sheets are welded together at 6 in. intervals with assumed crimping factor $s' = 0.008$ in./kip.

2.4.2 Description of Partition Type 2

This partition is similar to partition 1 in every respect except that it is welded to the edge member on one face only. Since in this case there is no need for a channel edge member, an angle L2 x 2 x 1/4 having a cross sectional area $A = 0.938$ in.^{2} is used. The sheet is welded to the angle at the center of the corrugations and hence $p = 6$ in. The crimping factor, s, between the sheet and the thicker material of the edge member is assumed 0.08 in./kip.

In addition to the foregoing data, the K factor required for the determination of the flexibility component due to bending of the corrugation profile calculated from Eq. 37 is given by

$$K = \frac{(d + 2h)(d^2 - 3\ell d + 3\ell^2)}{12\ h\ d^2} = \frac{(6+3)(6^2 - 3 \times 3 \times 6 + 3 \times 3^2)}{12 \times 1.5 \times 6^2} = 0.125$$

2.4.3 Description of Partition Type 3

This partition is identical to partition 2 except in the welding detail of the sheets to the edge members. Instead of welding the sheets at the center of the corrugations they are welded at the toes of the corrugations. Thus, the welding pitch p = 3 in. and the K factor required for determining C_3 is calculated from Eq. 38.

$$K = \frac{(2\ell + 3h)}{12(\ell + 6h)} = \frac{2 \times 3 + 3 \times 1.5}{12(3 + 6 \times 1.5)} = 0.073$$

2.4.4 Calculation of Shear Flexibility of Partitions

The shear flexibility of partitions 1, 2 and 3 described above, have been calculated from the basic expressions derived in Sect. 2.3. and are fully worked out in Table 1.

2.4.5 Analysis of Clad Frame

An elastic analysis of the bare frame was carried out to obtain the flexibility coefficients. Having thus generated matrix D (15,15) the program segment presented in subsection 2.2.1 was used to compute the restraining forces, net forces on the frame, shear forces on the partitions and the actual drift of the frame associated with the three shear flexibilities of the partition calculated in Table 1. A sample output for C = 5.76 x 10^{-3} in/kip is given in

Table 2. The results of the analysis are presented diagrammatically in Figs. 8-10. Each of Figs. 8, 9 and 10 shows the total applied shear at various floor levels and the share of this shear taken by the partitions. Each diagram also shows the maximum shearing force in the partition. These diagrams correspond to partition flexibility C equal to 7.91 x 10^{-3} in./kip, 5.76 x 10^{-3} in./kip and 3.67 x 10^{-3} in./kip respectively. They illustrate how relatively more flexible partitions attract less load. The maximum shear loads carried by the three partitions are 35.8 kip, 43.1 and 54.6 kip respectively. Figure 11 shows the deflected shape of the frame corresponding to the four cases. Curve A represents the bare frame behavior. As indicated the drift index at the roof level = 5.87/(15 x 12 x 12) = 0.0027. Curves B, C and D represent the behavior of the clad frame associated with values of C = 7.91 x 10^{-3}, 5.76 x 10^{-3} and 3.76 x 10^{-3}. The corresponding drift indexes are 0.0014, 0.0011 and 0.0009, which are one half, 0.42 and one third of the bare frame drift index.

It will be recalled that the first two drift indexes are associated with partitions where the sheeting is welded to the edge members on one face only. Consideration of expenses of sheet welding details and the resulting reduction in drift may make the designer be satisfied with welding the sheets on one face only provided the strength of the partition is not governed by the strength of the welded connections. Further, by welding the sheeting on both faces a partition can become so stiff that it may attract a share of the applied load in excess of its carrying capacity. In such a case resorting to the simpler welding details may provide the answer to the problem.

2.4.6 Check of Approximate Method

The approximate method proposed in subsection 2.2.2 will now be examined in the light of the results obtained for the design example discussed in Sect. 2.4.

The shear flexibility of columns, C_c, has been calculated from Eq. 9 and is listed below for the stories in the upper half of the frame.

Story No.	C_c (in./kip)
15, 14	9.65×10^{-3}
13, 12	3.95×10^{-3}
11, 10	2.54×10^{-3}
9, 8	1.86×10^{-3}

In the analysis of the design example, a value of $C = 7.91 \times 10^{-3}$ in./kip resulted in reducing frame drift exactly to one half. According to Eqs. 16 and 17 this can only happen if the flexibility of the columns, C_c, is the same as that of the partition. Scanning the above table would indicate that the shear flexibility of columns closest to this value lies between stories number 13 and 14 which are at about one fifth the frame height from the top. The average column flexibility of these two stories is 6.8×10^{-3} in./kip. Considering this value and according to Eqs. 16 and 17 the flexibility of the partition that would be required to reduce the drift to one third will be $0.5\ (6.8 \times 10^{-3}) = 3.4 \times 10^{-3}$ in./kip. This value corresponds to a value of 3.67×10^{-3} in./kip obtained in the design example. Also, the flexibility of the partition to reduce the drift to 0.42 of the bare frame value will be 0.73 (6.8×10^{-3}) = 5.0×10^{-3} in./kip. This corresponds to a value of 5.76×10^{-3} in./kip obtained in the example.

It follows that for the example by considering the flexibility of the columns at about one fifth the height of the frame from the top, Eq. 16 gives a good estimate of partition flexibility that would be required to reduce the drift of the frame to any prespecified fraction of the bare frame value.

2.5 Strength of Structural Partitions

So far emphasis has been placed on the stiffness of structural partitions with little reference to their strength. Consideration of the shear strength of a partition has to go hand in hand with consideration of its stiffness. This is because it is easy to choose a partition which would reduce the frame drift to within a prescribed limit and forget that due to its excessive stiffness it might not be capable of sustaining the share of load it attracts.

On examining the strength of a partition all the possible modes of failure must be considered. The design load should then correspond to the least calculated strength reduced by a reasonable load factor.

Failure of the partition can take place as a result of one or more of several reasons. These include tearing of the sheet at the edge member fasteners, breaking of the seam fasteners, yield of the sheet material and instability of the edge members. The latter is a more probable cause of failure especially for the horizontal edge members which are anticipated to span bay widths on the order of 30 ft. In considering the stability of such edge members it should be remembered that the strut has a length equal to half the total length of the edge member and that it is subjected to a uniformly varying axial load as explained earlier in subsection 2.3.3.

Another strength criterion which needs to be investigated is the shear buckling of the sheeting. This has been suggested as a design criterion by the investigators at Cornell University.[37] Corrugated shear panels, however, possess appreciable post buckling strength. Buckling of the most heavily loaded partitions near the bottom of the building will only cause a reduction in the stiffness of those partitions with insignificant effect on the overall behavior of the clad frames.

2.6 Strength of Clad Frames

In addition to the stiffening effect of the partitions, they add to the load carrying capacity of the frames. In allowable stress design the additional capacity corresponds to the share of the applied loads taken by the partitions. In maximum strength design the additional capacity results from two sources, the share of the applied loads taken by the partitions and at high loads, the larger capacity of the frame itself due to smaller $P\Delta$ effects.

3. TEST SETUP AND PROCEDURE

3.1 Introduction

The value of any theoretical study is greatly increased if it is correlated with experimental results. Further, the store of knowledge about actual behavior of structures is so meagre that any reliable tests on a large scale building will by itself, be of value and interest to engineers.

A large scale test building was constructed for the purpose of studying and isolating the various structural interactions which occur in a typical multi-story building. The building consists of unbraced steel frames, concrete floors and partitions made of light gage corrugated steel sheets. In this chapter, the building and the experimental capabilities it possesses will be discussed in detail. Also some aspects of planning, design, loading arrangement, fabrication, erection and instrumentation of the building will be given.

Description of the tests carried out on the test building in the elastic range, and discussion of the results in the light of the theory presented in Chapter 2 will be presented in Chapter 4.

Maximum load capacity tests conducted on individual clad frames are reported in Chapter 5.

3.2 Planning of Test Building

At a very early stage in planning of the test building there were many basic questions to be answered. For example a decision was required to establish how many stories, bays and frames the test building should have. Also, the overall dimensions had to be decided upon. To expedite these and many other similar decisions the following criteria were established:

1. No attempt need be made to exactly model the test building after an actual building with respect to dimensions and loads. Instead, it should be representative of a portion of a typical high-rise office or apartment building employing unbraced multi-story steel frames, composite steel-concrete floor systems and some permanent interior partitions. Further, it should be designed in accordance with current specifications and building codes where applicable as a building in its own right.
2. The test building should be large enough to employ standard and rolled steel beam and column sections in the frames.
3. The concrete floor should be designed so that it can easily be made either composite or non-composite with the frame beams.
4. The structural partitions should be constructed of available materials and provided with simple connections to the frame members so that they may be installed or removed at will.
5. The dimensions of the test building should be such that it will fit within the flexure test bed area in the

laboratory and not interfere with the operation of the overhead traveling crane.

6. The load required to test the building to its maximum capacity should not exceed the capacity of the available laboratory test facilities. Also, the loading condition should be pertinent to the problems involved in the design of high-rise buildings.
7. The test building should be designed and instrumented so that under load, the effects of the various components on the strength and stiffness of the building can be evaluated separately or in combination.

3.3 Description of Test Building

In order to fully understand the reasons for some of the details provided in the building and appreciate the experimental capabilities it provides it will be necessary to list the specific factors to be investigated. These are:

1. The effect of composite steel beam-concrete floor.
2. The effect of composite beam-column connections.
3. The effect of structural partitions

The three main structural elements of the building are described below:

3.3.1 Steel Framing

The test building consists of three parallel unbraced steel frames spaced 5'-4" apart. Each frame has two unequal bays of 10'-0" and 5'-0" and two equal stories with story height of 5'-0". A dimensioned sketch of the steel framing is shown in Fig. 12. A photograph of the building with the concrete floors is shown if Fig. 13. The frames are supported on pin bases at mid story height. This is in accordance with the assumption that the test building represents a portion within the height of a high-rise building where points of inflection usually occur very close to the mid points of the columns. The three frames are braced together for out of plane stability. Each frame is made of six subassemblages. Three typical subassemblages forming one story are shown in Fig. 14. The joints between the subassemblages allow either pinned or rigid connections. In addition to simplifying erection and allowing the replacement of damaged parts of the frames, this arrangement enables variations in lateral frame stiffness to be made. The steel framing is fabricated of ASTM A-36 rolled steel sections. The columns are W6x12 and the beams are S5x10 sections. MC3x7.1 sections are used for the floor beams. All the steel used for fabricating the frames was obtained from the same heat in order to reduce changes in cross-sectional and material properties.

3.3.2 Concrete Floor Slabs

Each floor slab was made up of nine pre-cast reinforced concrete panels. The six panels in the 10 ft. bay were each 5 ft. 3 1/2 in. (perpendicular to the frames) by 5 ft. 2 3/4 in. (parallel to the frames.) Similarly the three panels in the 5 ft. bay were 5 ft. 3 1/2 in. by 4 ft. 11 1/2 in. Each panel was 1.5 in. thick, with a 1 1/2 in. x 3/8 in. steel bar cast into the slab around the perimeter. Figure 15 shows a photograph of a panel before casting the concrete.

Shown on the centerline of the panel are 1.5 in. internal diameter pipe sleeves spaced at about 6 in. apart. These sleeves were provided to accomodate grouting of 1.2 in. shear connectors welded to the flanges of the frame beams. Reinforcement consisted of one layer of 4 in. x 4 in. wire mesh with 0.25 in.ϕ wire. Figure 16 shows a close-up view of a number of the shear connectors after erection. The shear connection was developed between the steel beam and the concrete floor by grouting inside the pipe sleeves with a fine concrete mix and using heavy square washers and nuts to secure the slabs. After grouting the floor panels could not be removed and the subsequent tests on the walls were carried out with the floor composite along the full length of the beams. Floor panels were provided with recesses with adequate clearance to accommodate the frame cloumns. Removable steel wedges were provided to engage the column faces with the concrete floor slabs. Figure 17 shows the wedge assembly and the manner in which the concrete floor slab could be made to bear against the column faces.

3.3.3 Structural Partitions

Partitions were fabricated of 24-gage (0.025 in. thick) ribbed steel sheets with a square profile 3 in. wide (in the plane of the partition) and 1.5 in. deep. Individual sheet widths were spot welded along the seams at about 4 in. spacing. The sheet was attached along its perimeter to light edge members made of 1.5 in. deep fabricated channel having 0.5 in legs and a cross-sectional area of 0.28 sq. in. The two sides running parallel to the corrugation generators were continuously welded to the vertical edge members. The two other sides were welded to the horizontal edge members; once along one face and then on both faces of the corrugations. Two partitions were provided for the 5-ft. and 10-ft. bays of each frame. Individual partitions were connected to the frame at the top corners and at intervals along the lower side. A corner connection was made up of two 0.25 in. thick plates, one welded to the partition and the other to the frame and both drilled to take a 0.5 in. bolt. Figure 18 shows a photograph of a typical corner connection. Use was made of the floor shear connectors to connect the lower side of the partition to the frame. The lower edge member was drilled and a nut was tightened on the threaded end of every other connector. Figure 19 shows one of these connectors. The figure also shows the recess in concrete slab provided to clear the column face. Figure 20 shows a photograph of a partition installed in the 5-ft bay. An overall view of the test building with partitions installed in the 10-ft. bays is shown in Fig. 21.

3.4 Design of Test Building

3.4.1 Steel Framing

The behavior of a typical multi-story frame under gravity and lateral loads may be described by the behavior of three main regions along its height.[5]

1. An upper region comprising about the top five to eight stories. In this region the design is controlled by the gravity loading condition. Second order $P\Delta$ effects are insignificant.
2. A transition region comprising a few stories immediately below the upper region in which the design is controlled by either the gravity or the combined gravity and lateral loading condition. The $P\Delta$ effect may or may not be significant.

3. A region comprising the middle and lower stories in which the design is controlled by the combined gravity and lateral load condition. The PΔ effects in this region are significant and should be considered in designs based on maximum strength.

It can be concluded, therefore, that it is not necessary to design a high-rise test building in order to investigate the integrated behavior of such buildings. The steel framing for the test building, although of a limited number of stories can be designed so that the behavior of similar stories anywhere in a taller building can be simulated simply by controlling the magnitudes of the column axial loads and shears at the boundaries.

3.4.2 Concrete Floor Slab

The concrete floor slab was designed primarily to fulfill the following requirements:

1. Maintain a realistic flexural stiffness relative to that of the frame beams and to provide a practical reinforcement ratio.
2. Enable the interactions between the frame and floor systems to be easily isolated and evaluated.
3. Support any gravity loads which may be applied during the testing.
4. Ease of handling during erection and testing.

3.4.3 Structural Partitions

So far as is known this is the first time that a test building with structural partitions made of light gage metal sheeting has been investigated. Consequently, there is no previous experience on which to base the design of the partitions and the connections between them and the bounding beams and columns. Therefore in designing the partitions, the following criteria were established:

1. A structural partition is, by definition, a structural element and as such should be adequately and permanently connected to the frames.
2. In order that a partition may behave primarily as a shear panel, it should be connected along its four sides. At the same time the connections should be detailed to clear the partition completely from the frame members, or at least provide some degree of in-plane flexibility in order to relieve the rather slender sheeting from undesirable in-plane compression resulting from bending of the frame members.

3.5 Loading Arrangement

The tests described in this report were carried out under lateral loads only. Nevertheless provisions had been made for applying gravity and lateral loads. Description of the two loading systems will be given.

The vertical loads were to be applied to the column tops by means of gravity load simulators connected to a system of loading beams as shown in the elevation view in Fig. 22. Each gravity load simulator has a maximum load capacity of 80 kips.[47] The load simulators were disposed in plan as shown in Fig. 23. This arrangement would have allowed the application of equal column loads up to a level of about 0.6 P_y, or column axial loads with varying gradients across the building.

The horizontal loads were applied to the frames by means of two screw jacks each of maximum load capacity of 40 kips. During the tests the three frames were loaded simultaneously through a rigid beam bolted to them at level 4, Figure 26. Two calibrated tension rods were inserted between the jacks and the loading beam for load measuring. The jacks operated against a longitudinal beam spanning between the laboratory columns. Figure 24 shows a photograph looking down on the loading apparatus. Also shown between the test building and the platform supporting the jacks are the three columns which held the dial gages.

The loading apparatus was planned and designed to fulfill the following requirements:

1. Maintain the relative position of the columns.
2. Distribute the applied vertical and horizontal loads equally to the three frames of the building.
3. Apply uniform column axial loads, P, up to a level of about 0.6 P_y. It was this requirement together with

the limitations imposed by the available loading capacity that decided the size of the column section adopted. In an actual building, the axial load level in a column varies along its height, being of the order of 0.5 - 0.6 P_y in the upper region and decreasing to 0.3 - 0.4 P_y in the lower region. Thus, under the gravity loading condition alone, the column axial loads could be adjusted so that the test building might represent any desired portion of a high-rise building.

4. Apply column axial loads with varying uniform gradients. Similar gradients across the building occur in actual buildings as a result of the overturning moments produced by the lateral loads. The gradient can be expected to increase towards the bottom of the building where the overturning moment has a relatively large influence.
5. Application of horizontal shear to the top of the test building to simulate the shear produced by lateral loads in a high-rise building.

Thus by adjusting the gradient of the column axial loads across the building and the magnitude of the horizontal shear, the test building could be made to represent any portion of a building under combined gravity and lateral loads.

3.6 Fabrication

The frames were fabricated of ASTM-A36 rolled steel sections; W6x12 for the columns and S5x10 for the beams. The required lengths of the sections used were ordered from one heat to avoid possible change in the cross sectional and material properties. The joints of the subassemblages, from which the frames were constructed, were welded by 1/4" all round fillet welds. The joints between adjacent subassemblages were bolted and all holes were drilled for 1/2" ϕ ASTM-A490 high strength bolts.

In order to minimize the sizes of the beams for the vertical loading apparatus, they were fabricated of ASTM-A50 high strength steel sections. The frames and the beams for the vertical loading apparatus were fabricated by an outside fabricator chosen on the basis of the lowest bid. The beams for the horizontal loading apparatus, the concrete floor panels and other items such as the bracing, frame pin bases, pedestals and the steel framing for the platform were fabricated in the laboratory shop. The partitions were fabricated by an outside firm which specializes in the fabrication of light-gage structural elements. Subassemblages enough to construct three frames, each of three stories, and floor panels for three floors were fabricated.

3.7 Erection

After the floor bases and the longitudinal anchor beams were layed and secured to the test bed, the pedestals were welded in their proper positions. Figure 25 shows a photograph of the test bed at an early stage of erecting the test building. The hinge bases of the frames were bolted to the pedestals after providing the necessary shims to make sure that they were all on the same level. On these bases, individual frames were erected with the bolts of the joints left loose. Vertical bracing between the frames and also braces in the floor planes were installed. Next, a set of measurements was taken at various levels to check the overall dimensions of the building and a transit was used to check the verticality of the columns. The necessary adjustments having been made, a new set of measurements was taken. The process was repeated until all the dimensions were correct and the columns vertical within the fabrication tolerance of 1/16" and then all the bolts of the joints were tightened.

After alignment of the test building, the vertical loading beams were installed and with minor adjustment they fitted in their proper places. Having thus established the level of the tops of the columns, the horizontal loading beams were installed in their proper places at the proper level.

Strain gages were next mounted on the frames at their pre-specified locations, labelled and hooked to the switch boxes provided to handle the large number of gages used. Finally, the frames were whitewashed with hydrated lime. Flaking of the whitewash during testing would indicate progression of yield if and where it occurred.

When the frame-floor tests started, the floor panels were placed in their proper places and temporarily secured to the frame beams while the positions of the shear connectors were marked by center punch through the sleeves cast in the floor panels. With the floor panels removed, the shear connectors were welded in their thus determined positions. Next the floor panels were replaced and shear connection along any length of the beams was obtained by tightening down nuts on the threaded ends of the connectors projecting through the sleeves.

When the tests on the building with partitions started, the self-contained shear panels were installed in the manner described in subsection 3.3.3.

3.8 Instrumentation

The instrumentation was designed to obtain strains from which a complete picture of the axial force and bending moment distributions could be drawn. The instrumentation was also designed to determine the magnitudes of the associated applied loads as well as their distribution among the frames. Electric resistance strain gages were used for measuring strains from which the stress resultants in all the members of the three frames were computed. Twenty strain gage stations on each frame were used. These were disposed along the beams and columns as shown in Fig. 26a. Each column strain gage station had four gages arranged on the flanges as shown in Fig. 26b. On the other hand each of the beam gage stations had six gages arranged as shown in Fig. 26c. The additional pair of gages attached to the web was provided to ensure accurate strain readings when the beams were made composite with the floors and the neutral axis was expected to move too close to the upper flange to make the gages attached there reliable. The upper pair of gages was attached to the underside of the top flange to avoid possible damage from the concrete slabs. Since the tests described in this report were carried out under lateral loads which resulted in very small axial strains, only bending moment distributions in the frames will be reported. A total of 96 strain gages per frame were used. This number alone almost exhausted the capacity of the available strain recording unit. Since during testing the strains induced in the three frames were to be recorded simultaneously, a special switch box assembly was constructed and all the strains were recorded automatically on punched tape by means of the available 100-channel strain recording unit. The punched tape was next converted onto computer cards which were used with other section and material properties as data to a specially developed computer program for data reduction.

The horizontal loads applied by the screw jacks were measured by means of two calibrated tension dynamometers inserted between the jacks and the loading beam.

In addition to strains, displacements were also recorded. Deflections of individual frames were measured at floor levels and mid-story heights by means of 1-in. travel dial gages. These gages were supported by three independent columns constructed for this purpose on the south side of the building. Figure 26a also shows the position of the dial gages along the height of the building. The results of deflection measurements at mid column height and at the loading beam level will not be quoted in this report as they do not contribute any new information. Only deflection readings at dial gage stations 1, 2 and 3 will be given. The dial gage reading at station A was used as the reference reading. The reading at station C was used to monitor the deflections applied to the frames by the screw jacks which were located at the level of station C.

4. TEST RESULTS - ELASTIC RANGE

4.1 Testing Procedure

In general the same procedure was followed in each of the tests. The initial dial gage and strain gage readings were recorded. Equal displacements were applied to the three frames. The applied displacements were indicated by dial gages placed at the upper floor level and when a suitable displacement had been reached the applied jack loads were noted and the corresponding deflection and strain values recorded. Successive displacement increments were then applied with the displacements and strains recorded each time. The maximum applied displacement was chosen to ensure that the frames were not loaded beyond the elastic range while the magnitude of the displacement increment in each test was chosen so as to provide a sufficient number of points on the load-deflection and load-strain curves. When the deflections and strain values had been recorded at the maximum applied displacement, the load was reduced to zero and the deflections were noted. It was observed in each test that the frames returned to their original unloaded position indicating that the building was behaving linearly and that the frames were not loaded beyond the elastic range. Each test was carried out twice to ensure the consistancy of the results.

4.2 Test Program

Three series of tests were carried out. The tests are numbered as follows:

1. Tests on the bare frames.
2. Tests on the frames with concrete floors.
   - (a) Floor made composite in the positive moment region under gravity loads.
   - (b) Same as in (a) but with the concrete floor in contact with the leeward faces of the columns.
   - (c) Floor made composite along the full length of beams.
   - (d) Same as in (c) but with the concrete floor in contact with the leeward faces of the columns.
3. Tests on the frames with composite floor and partitions.

   In this series a number of tests were carried out on the frames with and without the concrete floors in contact with the cloumn faces. Since no significant differences were observed the term composite floor will then mean that the frame beams are made composite along their full length and are in contact with the column faces.
   - (a) Partitions in the 5 ft-bay with the sheet welded to the edge members on one face of the corrugations only.
   - (b) Same as in (a) but with the sheet welded to the edge members on both faces of the corrugations.
   - (c) Partitions in the 10-ft. bay with the sheet welded to the edge members on one face of the corrugation only.
   - (d) Same as in (c) but with the sheet welded to the edge members on both faces of the corrugations.
   - (e) Partitions in both the 5-ft. and 10-ft. bays with the sheet welded to the edge members on both faces of the corrugations.

4.3 Control Tests

Control tests were done to determine the cross-sectional and material properties of the S5x10 and W6x12 steel sections used for the frame beams and columns of the test building.

4.3.1 Cross-Sectional Properties

The cross-sectional dimensions were measured by a micrometer and the actual properties were compared to their corresponding nominal values given in the manual of steel construction. The results for the S5x10 section used for the beams and the W6x12 section used for the columns are given in Table 3.

4.3.2 Tension Coupon Tests

Two coupons cut off the flanges and two off the web of each of the beam and column sections were tested in 300 kips screw type testing machine. The coupon shape and the testing procedure conformed with those described in ASTM Standard Specification A370. The load and the associated strain along an eight-inch gage length were measured and plotted by means of an extensometer and a low magnification automatic stress-strain recorder. The results thus obtained were compared to the load-strain relation obtained from the dial gage readings. Close agreement between both was obtained. However, due to the larger strain scale used for the dial gage readings, and hence the higher degree of accuracy, these readings were used to calculate the modulus of elasticity, E, and the other properties listed in Tables 4 and 5.

4.3.3 Beam Control Tests

A beam control test was performed on each of the frame beam and column sections to determine the moment-curvature relationship. From this relationship the average bending rigidity, EI, and the plastic moment, M_p, for both sections were obtained.

The tests yielded the following results:

For S5x10, EI = 354480 kip.in.^{2} and M_p = 209 kip.in.

For W6x12, EI = 656609 kip.in.^{2} and M_p = 296 kip.in.

Allowing for measured values of I = 13 in.^{4} for S5x10 and I = 22.2 in.^{4} for W6x12 the corresponding values of E are 27300 ksi and 29600 ksi respectively.

The above values have been used in the theoretical predictions.

4.3.4 Stub Column Test

A stub column test was performed on the frame column section (W6x12) to determine the average strength of the whole section. The specimen was 18 in. long. This length is equal to the minimum length of 3d and slightly less than the maximum length of 20 r_y specified in the Guide to Design Criteria for Metal Compression Members. Eight strain gages attached at the corners of two sections were used for alignment purposes while the shortening in a gage length of 10 in. was meausred by two dial gages attached at the middle of the flanges and reading in divisions of 0.0001 in. The value of EA was found to be equal to 101.700 kips. Allowing for the measured area A = 3.44 in.^{2} gives a value of E which is identical to that obtained from the beam control test.

4.4 Test Results

Detailed results of the deflection measurements are given. As to the strain measurements, it is impracticable to give the full results. However, by using a specially developed computer program for data reduction the strain readings were converted into bending moments and it is in that form the results are reported.

4.4.1 Deflection Measurements

The deflections at the first and second floor levels and at the mid column height of the upper story will be referred to as frame drift at deflection station 1, 2, and 3 respectively. (See Fig. 26)

The load deflection relations at these three stations of the bare frames are given in Fig. 27. Similar relations of the frames with composite floors are presented in Figs. 28-31. These figures shows the results of tests 2a, 2b, 2c, and 2d respectively. The load-deflection curves of the frames with composite floors and structural partitions are presented in Figs. 32-36. These figures show the results of tests 3a, 3b, 3c, 3d and 3e respectively. (See Art. 4.2) Loads shown in Figs. 27 to 36 are for an individual frame.

4.4.2 Strain Measurements

All the moments measured during tests 1-3e are given in Table 6. The moments are expressed in kip in. in terms of the applied lateral load (kip). The sign convention used is that moments at the ends of members are positive when clockwise. For convenience and ease of correlation between the experimental results and their corresponding theoretical values, bending moment diagrams of the bare frames, frames with composite floors along the full length of the beams, and frames with the various partitions used are also given. These bending moment diagrams are presented in Figs. 37-43. Indicated on the moment diagrams are the calculated values. The moments are plotted on the tension side of the members.

4.5 Predicted Behavior of Clad Test Building

Figure 44 shows a frame in the test building having a structural partition in the middle story. Under the applied load H, the frame will drift and will have horizontal displacements Δ_1, Δ_2, and Δ_3 at the three levels indicated. Due to the presence of the partition a restraining force H' will be developed at level 2 while a reactive restraining force H' is developed at level 1.

For the frame the actual displacements are given by:

$$\Delta_1 = H\delta_{14} - H'\delta_{12} + H'\delta_{11} \quad (49)$$

$$\Delta_2 = H\delta_{24} - H'\delta_{22} + H'\delta_{21} \quad (50)$$

$$\Delta_3 = H\delta_{34} - H'\delta_{32} + H'\delta_{31} \quad (51)$$

From Eqs. 49 and 50 the relative displacement between levels 1 and 2 is given by:

$$\Delta_2 - \Delta_1 = H(\delta_{24} - \delta_{14}) - H'(\delta_{22} + \delta_{11} - \delta_{21} - \delta_{12}) \quad (52)$$

For the partition and with reference to Fig. 44, the relative displacement between levels 1 and 2 is given by:

$$\Delta_2 - \Delta_1 = C\ H' \quad (53)$$

Equating the relative displacement in Eqs. 52 and 53 leads to:

$$H' = \frac{(\delta_{24} - \delta_{14})}{(C+\delta_{22} + \delta_{11} - \delta_{21} - \delta_{12})} H \qquad (54)$$

Having thus determined the force H' due to the presence of the partition, the actual displacements are found by back substitution in Eqs. 49, 50 and 51.

The flexibility coefficients appearing in these equations have been found from an elastic analysis of the frame under 1 kip loads applied individually at 1, 2 and 4. In this analysis, allowance was made of the composite floors. The resulting displacements (10^{-3} in.) are given below.

$\delta_{11} = 11.3815$	$\delta_{21} = 19.0842$	$\delta_{31} = 20.2006$	$\delta_{41} = 20.1780$
$\delta_{12} = 19.0842$	$\delta_{22} = 49.0252$	$\delta_{32} = 62.7331$	$\delta_{42} = 62.1596$
$\delta_{14} = 20.1780$	$\delta_{24} = 62.1596$	$\delta_{34} = 89.3835$	$\delta_{44} = 90.7559$

Substituting for the flexibility coefficients in Eq. 54,

$$H' = \frac{41.9816 \times 10^{-3}}{C + 22.2387 \times 10^{-3}} H \qquad (55)$$

Values of C for the partitions in the 5 ft. and 10 ft. bays for the two bases of welding the sheet to the edge members on both faces of the corrugations and on one face only are calculated in Table 7. The corresponding forces provided by the partitions and the associated frame deflections at levels 1, 2 and 3 are calculated below.

4.5.1 Partitions in 5 ft. Bay - Corrugations Welded on Both Faces

$$H' = \frac{41.9816x10^{-3}}{(23.78+22.2387)10^{-3}} H = 0.91 H$$

$$\Delta_1 = 10^{-3}H[20.1780-0.91(19.0842-11.3815)] = 13.0x10^{-3}H \text{ in.}$$

$$\Delta_2 = 10^{-3}H[62.1596-0.91(49.0252-19.0842)] = 34.9x10^{-3}H \text{ in.}$$

$$\Delta_3 = 10^{-3}H[89.3835-0.91(62.7331-20.2006)] = 50.7x10^{-3}H \text{ in.}$$

4.5.2 Partitions in 5 ft. Bay - Corrugations Welded on One Face

$$H' = \frac{41.9816x10^{-3}}{(72.23+22.2387)10^{-3}} H = 0.445H$$

$$\Delta_1 = 10^{-3}H[20.1780-0.445(19.0842-11.3815)] = 16.8x10^{-3}H \text{ in.}$$

$$\Delta_2 = 10^{-3}H[62.1596-0.445(49.0252-19.0842)] = 48.8x10^{-3}H \text{ in.}$$

$$\Delta_3 = 10^{-3}H[89.2835-0.445(62.7331-20.2006)] = 70.5x10^{-3}H \text{ in.}$$

4.5.3 Partitions in 10-ft Bay - Corrugations Welded on Both Faces

$$H' = \frac{41.9816x10^{-3}}{(9.62+22.2387)10^{-3}} H = 1.32H$$

$$\Delta_1 = 10^{-3}H[20.1780-1.32(19.0842-11.3815)] = 10.0x10^{-3}H \text{ in.}$$

$$\Delta_2 = 10^{-3}H[62.1596-1.32(49.0252-19.0842)] = 22.7x10^{-3}H \text{ in.}$$

$$\Delta_3 = 10^{-3}H[89.3835-1.32(62.7331-20.2006)] = 33.2x10^{-3}H \text{ in.}$$

4.5.4 Partitions in 10 ft. Bay - Corrugations Welded on One Face Only

$$H' = \frac{41.9816x10^{-3}}{(30.89+22.2387)10^{-3}} H = 0.79 H$$

$\Delta_1 = 10^{-3}H[20.1780-0.79(19.0842-11.3815)] = 14.0x10^{-3}H$ in.

$\Delta_2 = 10^{-3}H[62.1596-0.79(49.0252-19.0842)] = 38.6x10^{-3}H$ in.

$\Delta_3 = 10^{-3}H[89.3835-0.79(62.7331-20.2006)] = 55.8x10^{-3}H$ in.

4.5.5 Partitions in Both the 5 ft. and 10 ft. Bays - Corrugations Welded on Both Faces

This case needs special consideration. Let the shear flexibilities of the partitions in the 5 ft. and 10 ft. bays be C_1 and C_2 and the forces provided by them be H_1' and H_2' respectively. The actual relative displacement between floor levels 1 and 2 will then be given by:

$$\Delta_2 - \Delta_1 = C_1H_1' = C_2H_2' \tag{56}$$

The total force provided by the partitions H' is given by:

$$H' = H_1' + H_2' \tag{57}$$

From Eqs. 56 and 57

$$H_1' = \frac{C_2}{C_1+C_2} H' \quad \text{and} \quad H_2' = \frac{C_1}{C_1+C_2} H' \tag{58}$$

From Eqs. 56 and 58

$$\Delta_2 - \Delta_1 = \frac{C_1C_2}{C_1+C_2} H' \tag{59}$$

Eq. 59 can be expressed as:

$$\Delta_2 = \Delta_1 = CH' \tag{60}$$

where C, an equivalent flexibility of the two partitions, is given by:

$$C = \frac{C_1 \; C_2}{C_1 + C_2} \tag{61}$$

Equating the relative displacement in Eqs. 60 and 52 leads to the expression for the total force provided by the partitions.

$$H' = \frac{(\delta_{21} - \delta_{31})}{(C+\delta_{22} + \delta_{33} - \delta_{23} - \delta_{32})} H \qquad (62)$$

Once this force is found the part taken by individual partitions may be found from Eq. 58. Substituting for individual partitions flexibility from Table 7 into Eq. 61 gives the equivalent flexibility of the two partitions.

$$C = \frac{23.78 \times 9.62 \times 10^{-6}}{(23.78+9.62)10^{-3}} = 6.87 \times 10^{-3} \text{ in/kip}$$

$$H' = \frac{41.9816 \times 10^{-3}}{(6.87+22.2387)10^{-3}} H = 1.44 H$$

of which H_1' is taken by the partition in the 5 ft. bay and H_2' is taken by the partition in the 10 ft. bay.

$$H_1' = \frac{9.62 \times 10^{-3}}{(23.78+9.62)10^{-3}} \times 1.44H = 0.414H$$

$$H_2' = \frac{23.78 \times 10^{-3}}{(23.78+9.62)10^{-3}} \times 1.44H = 1.026H$$

$$\Delta_1 = 10^{-3}H[20.1780-1.44(19.0842-11.3815)] = 9.1 \times 10^{-3}H \text{ in.}$$

$$\Delta_2 = 10^{-3}H[62.1596-1.44(49.0252-19.0842)] = 19.0 \times 10^{-3}H \text{ in.}$$

$$\Delta_3 = 10^{-3}H[89.3835-1.44(62.7331-20.2006)] = 28.2 \times 10^{-3}H \text{ in.}$$

The forces provided by the partitions and the actual drift of clad frames calculated in this section will be used in the subsequent analysis of the results.

4.6 Analysis of Results

In the analysis of the test results and the assessment of their significance attention will be concentrated on the following two aspects:

1. The difference in structural behavior between completed buildings and idealized bare frames.
2. Agreement between observed behavior and that predicted according to the theory proposed in this report.

4.6.1 Bare Frames

Using the properties of the beam and column sections determined from the control tests the frame was subjected to a first order elastic analysis. The calculated deflections at deflection stations 1, 2 and 3 are shown together with the measured values in Fig. 27. It is seen that the frames are slightly more flexible than predicted. This additional flexibility must be due to lack of complete rigidity in the frame joints at mid column heights and some slip at the upper and lower boundaries. Evidence of that slip at the boundary may be seen in the bending moment diagram of the bare frame shown in Fig. 37 where the column moments in the upper and lower stories exhibit some form of shear lag. Apart from this effect the measured moments seem to be in good agreement with the calculated values. The accuracy of the results has been checked by the consideration of the moment equilibrium of all the joints and the shear equilibrium of the three stories. The out of balance moments at the various joints ranged between 2% and 7% of the applied joint moments, and the out of balance shear in the three stories ranged between 2% and 6% of the applied shear.

4.6.2 Frames with Composite Beams

In test 2a where the frame beams were made composite in the positive moment regions under gravity loads (the middle part of the beams of about 0.6 of the span length), almost no change in the behavior of frames were recorded. This can be seen by comparing the deflections measured during this test and presented in Fig. 28, and the deflections of the bare frame shown in Fig. 27. It can also be seen by comparing the corresponding moments in Columns 2 and 3 of Table 6.

In test 2b where the same floors were engaged to the column faces a noticeable reduction in the building drift was recorded. This reduction amounted to about 15% of the bare frame drift.

In test 2c where the beams were made composite along their full length, a slight increase in the building stiffness was noticed This increase, however, was still less than that recorded during the 2b where the beams were made composite only in the positive moment regions but with the floors engaged to the column faces. The corresponding theoretical deflections showed in solid lines in Fig. 30 were calculated on the basis of the frames having composite beams along their full lengths and allowing for the gaps in the floor near the columns. The effective slab width considered in the calculations corresponds to that specified by the AISC; 27 in. for the beams in the 10 ft. bay and 5 in. for the beams in the 5 ft. bay. The modular ratio was taken equal to 9. On this basis, the calculated and the measured deflections are in a very good agreement. Also the calculated bending moments shown in Fig. 38 differ from the measured values by nearly the same amount found in the earlier test on the bare frames. Since, as mentioned before, all the tests on the building with partitions were conducted with the beams made composite as in test 2c, the frame behavior during this test, rather than that of the bare frame, is considered as the basic behavior when comparing the relative effects of the partitions.

In test 2d, which was similar to test 2c except that in the former test the floors were engaged to the column faces, the building showed a noticeable increase in stiffness. This increase in stiffness which amounted to about 15% is the same as that recorded in test 2b where the beams were made composite only in the positive moment regions. It seems, therefore, that as long as the concrete floors are engaged to the column faces, composite beams have the same stiffening effect regardless of whether the beams are composite along their full lengths or part of their lengths.

4.6.3 Frames with Structural Partitions

The deflections of the clad frames with the different partitions and attachments used have been calculated in Section 4.5. These calculated deflections are plotted with their corresponding measured values in Figs. 32-36. It is seen from these figures that an extremely good agreement exists between the measured and calculated values for all the cases considered of the partitions in the 5 ft. bay, the 10 ft. bay and both bays, and also with the sheets welded to the edge members on one face and both faces of the corrugations.

In order to show the appreciable effect structural partitions have on the stiffness of the building and be able to draw a proper comparison between the behavior of the clad frames and that of the basic frame the deflections of the basic frame (test 2c) and the deflections of the frames with partitions (tests 3a-3e) are summarized in Table 8. The deflections in this table are expressed in terms of the applied load in units of 10^{-3} in. Table 8 also includes the relative displacements between levels 1 and 2. Since the partitions were installed between these two levels only, these relative displacements will be considered in evaluating the stiffening effect of partitions. The reduction in drift given in the last column of Table 8 is expressed as a percentage of the drift of the basic frame (test 2c). It can be seen that when the partitions were installed in the 5 ft. bay and with the sheeting welded to the edge members on one face of the corrugations the drift was reduced by 23.8%. With the partitions still in the 5 ft. bay but with the sheeting welded to the edge members on both faces of the corrugations the reduction in drift was more than doubled. When the partitions were installed in the 10 ft. bay with the sheeting welded to the edge members on one face only the drift was reduced by 41.4%. When the same partitions were welded on both faces of the corrugations the reduction in drift was further increased to 67.3%. When the partitions were installed in both bays with the corrugations welded on both faces a drastic reduction in drift of 76.3% was recorded.

The bending moments in the clad frames with the five different partition arrangements used have been calculated. The calculations were based on the forces provided by the partitions calculated in section 4.5. Considering the complexity of the completed test building and the lack of complete rigidity in the joints and the slip at the boundaries noticed during earlier tests on the bare frames and the frames with composite floors, the calculated and observed moments are in reasonable agreement. Also both moments bear the same general feature of reductions in the column moments of the middle story where partitions were installed. When the partitions were relatively stiff as in tests 3b, 3d and 3e the reductions were so great that some of the column moments reversed sign.

In order to check the results and have a measure of the share of the applied load resisted by the partitions and the net loads on the frames, the shear resisted by the columns of each of the three stories of the test building were calculated from the moments measured during the various tests and in each case compared to their corresponding theoretical values. The results are presented in Table 9. All the shears entered in this table are expressed in terms of the applied lateral load. The theoretical shears are based on the calculations in section 4.5 while the measured shears are based on the measured moments presented in Table 6. The measured shears were calculated according to the following formula:

$$\text{First story shear} = \frac{1}{21}(M_1 + M_5 + M_9) \qquad (63)$$

$$\text{Second story shear} = \frac{1}{42}(M_2 + M_3 + M_6 + M_7 + M_{10} + M_{11}) \qquad (64)$$

$$\text{Third story shear} = \frac{1}{21}(M_4 + M_8 + M_{12}) \qquad (65)$$

where M_1- M_{12} are the column moments recorded at strain stations number 1-12, and 21, 42, and 21 are the corresponding moment arms in inches.

A study of the shear forces in Table 9 shows that the shears in the first and third stories recorded during all tests are very close to the applied lateral load. The maximum difference between the observed and calculated shears is less than 7% and in most cases much less than this figure.

The shears in the second story are of special interest. Only when partitions were installed did the shearing force in this story show any difference to the shearing forces in the first and third stories. In test 3a where the partitions was installed in the 5 ft. bay and with the sheeting welded on one face of the corrugations the observed shear resisted by the frames was about 60% of the applied lateral loads and the remaining 40% resisted by the partitions. These correspond to calculated values of about 55% and 45% respectively. With the partitions still in the 5 ft. bay but with the sheeting welded on both faces of the corrugations the observed shear resisted by the frames was about 17% of the applied loads while the reamining 83% was taken up by the partitions. These correpond to calculated values of 9% and 91% respectively. When the partitions were installed in the 10ft. bay and with the sheeting welded on one face of the corrugations the observed shear resisted by the frames was about 31% of the applied lateral load while the remaining 69% was taken up by the partitions. These correspond to calculated values of 21% and 79% respectively. When the same partitions were welded on both faces of the corrugations the observed shear resisted by the frames was about 22% of the applied load and of reversed sign. The partitions then carried a shearing force of about 22% more than the applied load. The corresponding calculated value is 32% more than the applied load. Finally when the partitions were installed in both bays, the observed shear resisted by the frames was about 33% of the applied shear load and of reversed sign. The partitions then carried a total load of 1.33 times the applied load. The corresponding calculated value is 1.44 times the applied load.

5. ULTIMATE STRENGTH TESTS

5.1 Testing Procedure

After completion of all the initial elastic tests described in this report, a pilot program was undertaken to determine the ultimate strength of the clad frames. Before testing, each frame consisting of steel columns and composite beams, was structurally isolated from the others, thereby providing three test frames. Two frames were tested with structural partitions installed. The third frame was tested without partitions and used for comparative purposes. Each frame was displaced at the top by a single horizontal screw jack in a manner similar to that shown in Figs. 13 and 22. No axial loads were applied to the columns. Only deflections of the frame and jack loads were recorded. No strain gage data from the frames was recorded. The ultimate strength tests were terminated only after significant distortion and tearing of the structual partitions had occurred. To prevent premature failure of the half-storys above and below the story containing a partition each frame had diagonal bracing placed in the 10-ft. bay. The diagonal bracing used was 1-in. diam. steel rods. Figure 45 shows the diagonal bracing in the bottom half story. The diagonal bracing in the top half story was similar.

The structural partitions used here were the same ones that had been used in the initial elastic tests and described in Chapters 3 and 4. It was found that after these tests, the partitions and frames were slightly distorted. In order to place the partitions back into the frames for the ultimate strength tests some cutting and fitting had to be done along the bottom of each partition where it connected to the composite beam. As a result the initial elastic stiffness of each partition was altered.

Since the correlation between elastic predictions and test results was already shown, the purpose of the ultimate strength tests was mainly to determine qualitatively the behavior of the integrated frame-floor-wall system under large lateral load. In addition the qualitative lateral load versus shear behavior of each partition was desired under large shear distortion.

5.2 Test Program

Three ultimate strength tests were performed, one for each frame. For frames 1 and 2 (Fig. 23) partitions were installed in the 10-ft. and 5-ft. bays respectively. The partitions used and the method of fastening them to the frames are described in Art. 3.3.3. The sheets were welded to the edge members on both faces of the corrugations as for tests 3b and 3d (Art. 4.2). The tests are numbered as follows:

Frame 1: Structural partition in 10-ft. bay only. (Similar to Fig. 21)

Frame 2: Structural partition in 5-ft. bay only. (Similar to Fig. 46)

Frame 3: No structural partitions (frame-floor system only)

5.3 Control Tests

Cross-sectional and material properties of the S5x10 and W6x12 steel sections are given in Art. 4.3. The average yield stress level of the 24 ga. steel sheet used in the partitions was 39.6 ksi.

5.4 Test Results

The load-drift behavior of Frames 1, 2 and 3 are shown in Figs. 47, 48 and 49 respectively. In each figure the measured horizontal jack load at station 4 is plotted against the horizontal drift of stations 1, 2 and 3 relative to the reference station A (Fig. 26). The irregular behavior of the frames as shown in Figs. 47 and 48 can be attributed to; (1) sudden slip between the partition edge member and the concrete floor slab (Fig. 19); (2) sheet distortion between seam fasteners, and (3) tearing of the sheets near the edge members.

Figure 50 shows the partition in the 10-ft. bay of Frame 1 after the ultimate strength test. Considerable crimping of the corrugations can be seen along the lower side where the edge member was fastened at small intervals to the floor system (Art. 3.3.3). Very little crimping of the corrugations occurred along the upper side where the edge member was fastened to the frame only at the two corners (Art. 3.3.3).

Considerable tearing, twisting and out of plane bending of one complete corrugation occurred almost exactly at the mid-point of the partition and is clearly evident in Fig. 50. The seam fasteners were closely spaced which prevented large distortions of the sheet between fasteners or significant movements at the fasteners.

Figure 51 shows the partition in the 5-ft. bay of Frame 2 near the ultimate load. Buckling of the sheet between seam fasteners can be seen. This type of sheet distortion as well as tearing and twisting of the corrugations was also evident at the ultimate load. The overall appearance of both the 5-ft. and 10-ft. partitions after the ultimate strength had been reached was similar.

5.5 Analysis of Test Results

The test results used to plot the curves in Figs. 47 to 49 were also used to obtain an indication of the lateral load versus shear behavior of the structural partitions in the test frames. The load-drift behavior of Frame 3 (Fig. 49) was assumed to be the same for all three frames in the absence of partitions. The increased lateral load obtained for Frames 1 and 2 therefore represents the shear force H' carried by the partition, providing that the partition does not restrain the flexural behavior of the beams and columns. Although the partitions were continuously connected to the composite beam along the lower edge it was assumed that the restraint provided to the beams was minimal.

Figure 52 shows the relationship between H' and the relative drift between levels 1 and 2 (Fig. 26) for the partitions in the 10-ft. and 5-ft. bays. The curves agree with the behavior suspected by previous investigators. Light gage steel structural partitions develop high initial shear stiffness. As a result, the partition will initially carry a significant share of the shear carried by the frame-floor-wall system. However, because the structural partition material is very thin it will begin to tear in the vicinity of the fasteners resulting in a sharp reduction in shear capacity. Evidently, as the shearing distortion increases, a redistribution of forces occurs, resulting in a stabilizing of the shear capacity. This was likely due to the development of tension field action in the structural partition.

6. SUMMARY CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

The research reported herein has investigated the structural interactions which occur between steel frames, composite steel-concrete floor systems and wall systems consisting of light gage ribbed or corrugated steel sheet partitions or cladding in multistory buildings. The design criterion for such buildings over about twenty stories and employing rectangular unbraced steel frames with moment resisting connections is usually drift at working loads rather than overall strength.[38] To design for drift by increasing only the frame member sizes (usually the beams) is somewhat uneconomical and not entirely rational. This procedure ignores the stiffening effect due to interactions which already exist between the steel frames and the floor and wall systems. Some of these interactions are structural in nature and some may be potentially structural. Significant differences were shown to occur between the calculated and actual stress resultants in the frames if these interactions are ignored. In fact some stress resultants can be completely reversed.

A theoretical and experimental investigation was undertaken at Fritz Laboratory to study the structural interactions which can and do exist between unbraced frame, floor and wall systems. In addition the study included the development of a method whereby the stiffening effects of these elements can be included in design for drift.

The frames considered in the investigation were rectangular unbraced steel frames with rigid connections. The floor system was assumed to be either solid concrete or concrete on metal deck both of which are made composite with the frame beams. The wall system considered was either interior structural partitions or curtain walls or both, made of light-gage corrugated or ribbed steel sheeting.

Studies of the frame-floor system interaction at Fritz Laboratory are still in progress and the results are reported elsewhere.[21, 48, 49] Apart from a brief investigation of the frame-floor interaction studies, this study focused mainly on the frame-wall interaction phase of the program.

An analytical treatment of the frame-wall system interaction was undertaken and a method suggested for considering the effect of structural partitions and cladding on frame behavior. The method is based on the flexibility coefficients of the steel frame and is applicable regardless of the material used for the wall system. The stiffening effect of the floor system was found to be easily included by replacing the influence coefficients of the steel frame alone by those of a steel frame having composite steel-concrete beams. The basis of the method of computing these coefficients is reported elsewhere.[21, 48, 49]

An approximate method for estimating the drift of a clad frame was also developed. The accuracy of this method was checked by comparison with the results of a design example using an exact procedure.

The results of an experimental investigation of a large size test building were presented. The test building was designed to explore the integrated behavior of multistory buildings under gravity, lateral and combined loading conditions. However, the investigation was limited to experiments with the lateral loading conditions only. The experiments which were carried out included tests on the bare steel frames, on the frames with composite concrete floor systems, and with structural partitions. Emphasis was placed on the stiffening effect of the structural partitions on the overall building behavior. The test results showed good agreement with predicted results in all cases.

Finally, the test building containing the composite floor system and structural partitions was tested to its maximum capacity under lateral loads only. The test was conducted in such a way that the shear force versus shear distortion relationship for the structural partitions within the test building could be obtained for large distortions.

6.2 Conclusions

The major conclusions of this investigation are divided into the two primary study areas: (1) frame-floor system interaction and (2) frame-floor-wall or frame-wall system interactions as follows:

6.2.1 Frame-Floor System Interaction

1. The analysis and the experimental results showed that significant interaction occurs between the steel frames and the composite floor system. The interaction is somewhat dependent on establishing full contact or bearing between the steel columns and the surrounding concrete floor when considering lateral loads.
2. Frame-floor system interaction can be considered by replacing the frame beams with a composite beam having the properties of the composite floor system, resulting in another equivalent frame.
3. This study showed that frame-floor system interaction resulted in smaller building drift under working loads than that obtained considering the frame alone.
4. Related studies reported elsewhere indicate that frame-floor system interaction significantly increases frame strength and stiffness. [21, 48, 49]

6.2.2 Frame-(Floor)-Wall System Interaction

1. The frame-(floor)-wall system interaction has a major effect on reducing building drift under working loads and should be considered in design.
2. Relatively simple and straight forward analysis and design procedures have been developed which can account for frame-wall system interaction and its effect on drift, as well as frame and wall system stress resultants.

3. There exists several fundamental differences between the previous work reported on roof and deck panels and their influence on building behavior, and the present work on structural partitions. Both have concerned the influence of light gage ribbed or corrugated steel panels on building behavior. However differences occur concerning the direction of shear resistance relative to the corrugation generators, the method of fastening the panels to edge members and the number and location of connections between the edge members and the frames.
4. Frame-wall system interaction in buildings can be easily considered in design by designating certain permanent interior or exterior partition or wall elements as structural partitions or structural cladding and proportioning them for the loads transmitted to them. The results will be two-fold: (1) the calculated stress resultants in the steel frames and the structural partition or cladding elements will be closer to reality, and (2) drift under working loads, being related to the actual interactions between the frame and wall systems will be more predictable.
5. Experiments on the large-size test building showed that the stress resultants in the steel frame are considerably different depending on whether the analysis considers the actual structural interactions between the frame and the floor and wall systems or ignores them. In fact some of the stress resultants can be of opposite sign.

6. The experiments also showed that frame-(floor)-wall system interaction substantially reduces building drift under lateral loads.
7. Light-gage structural partitions can be effective for drift control at working loads. However under increasing loads they lose considerable strength due to distortion and tearing of the sheets along the perimeter and at the seam fasteners. It is likely that structural partitions can be designed for drift control providing the frame and floor systems are adequate for strength if the capacity of the structural partitions is exceeded.

6.3 Recommedations

The following recommendations are based on the investigation reported herein. Many of them reinforce the recommendations contained in the Cornell report.[37]

1. Research in progress at Fritz Laboratory on frame-floor system interaction is investigating the moment capacity and ductility of composite beam-to-column connections for use in unbraced frames with fully welded steel beam-to-column connections. Further research is required on composite connections where the steel beam-to-column connection is of the simple shear or semi-rigid type.
2. The substantial drift reductions achieved by considering frame-floor-wall system interactions in multistory buildings warrants a serious study of many practical factors involved

such as (1) the best detail at a column slab interface to achieve full bearing, (2) practical and effective connections between light-gage structural partitions and supporting members, (3) the most efficient profiles for light-gage structural partitions, and (4) construction details to ensure that structural partitions or cladding can not easily be removed while the building is in service.

3. Further research is required on the ultimate strength behavior of multistory frames with structural partitions. The research reported herein as well as that completed by Oppenheim(42) suggests that partitions designed for drift control may not contribute to overall building strength. The light-gage structural partitions considered in this investigation have high shear stiffness and were designed to resist a considerable share of the story shear below the working load level. However under high shearing distortion, tearing of the sheet, and buckling and distortion between fasteners substantially reduced the shear capacity of the structural partition. In this case the frame-floor system alone would be designed to resist the maximum combined gravity and lateral loads. Further research should explore the relationship between frame-structural partition stiffness ratios for low load levels and frame-structural partition strength and ductility ratios for large load levels and large values of drift. The effect of structural-partitions on overall building failure modes needs investigation. All of these studies should include the P- Δ effect.

4. Further large scale three dimensional tests of steel framed buildings are required which would investigate the influence of structural partitions at high load levels where P-Δ effects and panel shear distortion are large. These tests would be designed to investigate the stiffness, strength and ductility ratios mentioned above, and the modes of failure. They would also assist in the development of analytical methods to predict the ultimate strength of buildings with integrated frame-floor-wall systems.

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8. NOMENCLATURE

A	=	Cross-Sectional area of edge members
$[A]$	=	Coefficient matrix
a	=	Width of partition (measured parallel to the corrugation generator).
A_1	=	Cross-Sectional area of horizontal edge members
A_2	=	Cross-sectional area of vertical edge members
$\{B\}$	=	Floor displacement vector
b	=	Length of partition
b_o	=	Average sheet width b/n_c
C_c	=	Shear flexibility of frame columns defined in Eq. (9)
C_1-C_5	=	Shear flexibility components of partitions
$[D]$	=	Flexibility coefficient matrix
$\{D\}$	=	Joint displacement vector
d	=	Pitch of corrugation
E	=	Modulus of elasticity
$\{F\}$	=	Load vector
G	=	The shear modulus
H	=	Applied lateral load
H'	=	Restraining force provided by partition
h	=	height of corrugation
I_c	=	Moment of inertia of column section
K	=	Factor depending on the geometry of the corrugation profile and the attachments to the edge members, defined in Eqs. 37 and 38.

[K]	=	The structure stiffness matrix
L	=	Story height
ℓ	=	Widths of crest of corrugation
m	=	Number of joints in frame
n	=	Number of stories of frame
n_c	=	Number of corrugation in partition = b/d
p	=	Pitch of edge fasteners
p_a	=	Pitch of fasteners along vertical edge members
p_b	=	Pitch of fasteners along horizontal edge members
p'	=	Pitch of seam fasteners
Q	=	Shearing force on partition
q	=	Shear stress in sheet
r	=	Relative flexibility factor, defined in Eq. 17
s	=	Local deformation at each edge fastener per unit load "crimping"
s'	=	Local deformation at each seam fastener per unit load
t	=	Thickness of sheet
U	=	Strain energy
u	=	Shear strain energy per unit volume
α	=	Factor depending on corrugation profile $= \frac{\text{developed length of corrugation}}{\text{projected length of corrugation}}$
γ	=	Shear distortion of partition
Δ	=	Actual drift of clad frame
Δ_b	=	Drift of bare frame
δ_{ij}	=	Flexibility coefficient; drift at floor level i due to a unit load at floor level j
ν	=	Poisions ratio

9. TABLES AND FIGURES

TABLE 1 SHEAR FLEXIBILITY OF PARTITIONS

Shear Flex. Component	Partition Type 1	Partition Type 2	Partition Type 3
$C_1 = \frac{2\alpha a(1+\nu)}{E\ b\ t}$	$= \frac{2x1.5x114(1+0.25)}{30000x346x0.0478}$ $= 0.86x10^{-3}$ in/kip	$= 0.86x10^{-3}$ in/kip	$= 0.86x10^{-3}$ in/kip
$C_2 = \frac{(b^3 + a^3)}{6\ EA\ b^2}$	$= \frac{(346^3 + 114^3)}{6x29000x0.875x346^2}$ $= 2.35x10^{-3}$ in/kip	$= \frac{(346^3 + 114^3)}{6x29000x0.938x346^2}$ $= 2.18x10^{-3}$ in/kip	$= 2.18x10^{-3}$ in/kip
$C_3 = \frac{144\ K\ h^3 \ell^2}{E\ t^3\ a\ b\ d}$	------	$= \frac{144x0.125x1.5^3x3^2}{30000x0.0478^3x114x346x6}$ $= 0.71x10^{-3}$ in/kip	$= \frac{144x0.073x1.5^3x3^2}{30000x0.0478^3x114x346x6}$ $= 0.41x10^{-3}$ in/kip
$C_4 = \frac{2\ s\ p}{b^2}(a + b)$	------	$= \frac{2x0.08x6}{346^2}(114 + 346)$ $= 3.70x10^{-3}$ in/kip	$= \frac{2x0.08x3}{346^2}(114 + 346)$ $= 1.85x10^{-3}$ in/kip
$C_5 = \frac{a\ p's'}{b\ b_o}$	$= \frac{144x6x0.008}{346x34}$ $= 0.46x10^{-3}$ in/kip	$= 0.46x10^{-3}$ in/kip	$= 0.46x10^{-3}$ in/kip
$C = \sum_1^5 C$	$3.67x10^{-3}$ in/kip	$7.91x10^{-3}$ in/kip	$5.76x10^{-3}$ in/kip

TABLE 2 OUTPUT FOR C = 5.76 x 10^{-3} in/kip

LEVEL	RESTRAINT FORCES
1	.63425E+01
2	.56535E+01
3	.38974E+01
4	.46068E+01
5	.23328E+01
6	.40134E+01
7	.24492E+01
8	.35271E+01
9	.27402E+01
10	.31753E+01
11	.15178E+01
12	.21759E+01
13	.64710E+00
14	-.90911E-01
15	-.15119E+02

NET FORCES ON FRAME	S.F. IN WALL
.8575484E+00	.6342452E+01
.1546506E+01	.1199595E+02
.3302632E+01	.1589331E+02
.2593187E+01	.2050013E+02
.4867166E+01	.2283296E+02
.3186640E+01	.2684632E+02
.4750844E+01	.2929548E+02
.3672939E+01	.3282254E+02
.4459768E+01	.3556277E+02
.4024704E+01	.3873807E+02
.5682184E+01	.4025588E+02
.5024126E+01	.4243176E+02
.6552901E+01	.4307886E+02
.7290911E+01	.4298795E+02
.2231872E+02	.2786922E+02

ACTUAL DISPLACEMENTS
.2519733E+01
.2483200E+01
.2414104E+01
.2322558E+01
.2204478E+01
.2072960E+01
.1918325E+01
.1749583E+01
.1560525E+01
.1355684E+01
.1132552E+01
.9006784E+00
.6562715E+00
.4081373E+00
.1605267E+00

TABLE 3 CROSS SECTIONAL PROPERTIES

Section / Property	S5x10		W6x12	
	Nominal	Measured	Nominal	Measured
A	2.94	2.95	3.54	3.44
d	5.00	5.064	6.00	6.07
b	3.004	2.990	4.00	4.023
t_f	0.326	0.329	0.279	0.278
t_w	0.214	0.210	0.230	0.208
I_x	12.3	13.0	21.7	22.2
S_x	4.92	5.13	7.25	7.32
r_x	2.05	2.10	2.48	2.54
Z	5.67	5.91	8.23	8.22
Shape Factor	1.14	1.15	1.13	1.12

TABLE 4 MATERIAL PROPERTIES OF S5x10

TEST NO.	LOCATION	σ_y(ksi)	σ_{ult}(ksi)	ϵ_y%	ϵ_{st}%	E(ksi)
1	FLANGE	34.755	62.889	--*	0.625	28444
2	FLANGE	33.882	66.652	--*	1.000	28199
3	WEB	46.247	72.210	--*	0.569	28735
4	WEB	42.738	72.388	0.15	1.306	28715

TABLE 5 MATERIAL PROPERTIES OF W6x12

TEST NO.	LOCATION	σ_y(ksi)	σ_{ult}(ksi)	ϵ_y%	ϵ_{st}%	E(ksi)
5	FLANGE	36.268	62.038	--*	1.662	29778
6	FLANGE	35.833	63.235	--*	1.687	30392
7	WEB	40.116	64.018	0.15	2.540	29716
8	WEB	39.858	61.909	0.14	2.675	31283

*Absence of definite yield.

TABLE 6 BENDING MOMENTS IN TEST BUILDING

Station No. \ Test No.	1	2a	2b	2c	2d	3a	3b	3c	3d	3e
1	5.4	5.2	5.6	5.6	5.6	5.8	5.6	6.8	7.6	6.8
2	4.7	5.0	4.1	4.8	4.5	2.8	-1.4	1.2	-1.2	-1.6
3	1.6	1.8	1.8	1.8	1.8	1.2	-3.2	-2.4	-6.7	-6.4
4	7.1	6.8	7.2	7.4	7.5	8.0	7.8	8.4	8.2	8.7
5	8.8	9.4	8.8	8.9	9.5	8.2	8.2	8.2	7.8	7.4
6	9.8	10.2	10.0	9.8	10.0	6.6	3.8	5.0	0.6	-0.4
7	11.8	11.6	10.8	11.6	10.8	7.4	3.2	4.8	1.0	-1.3
8	7.2	7.6	7.6	7.0	7.6	7.4	6.2	7.0	6.6	6.2
9	6.0	5.4	5.2	5.2	4.8	5.6	5.9	5.0	4.3	5.4
10	6.3	6.0	6.4	6.2	6.7	3.6	2.5	2.2	-1.0	-1.8
11	7.2	6.8	8.0	7.2	7.6	3.8	2.4	2.4	-2.0	-2.2
12	6.2	6.0	5.6	5.8	5.8	4.8	6.4	5.6	5.4	5.2
13	-9.8	-10.8	-9.8	-10.6	-10.6	-9.8	-6.0	-7.0	-3.8	-3.4
14	-8.4	-8.8	-10.2	-8.6	-10.2	-7.2	-5.8	-6.8	-3.6	-2.8
15	-10.7	-10.0	-8.8	-9.3	-8.4	-7.8	-3.2	-5.6	-2.0	-1.2
16	-12.2	-11.0	-11.4	-11.0	-11.2	-7.2	-4.8	-6.8	-4.2	-3.4
17	-10.6	-10.4	-10.0	-10.8	-10.7	-9.6	-6.0	-7.2	-5.2	-3.8
18	-8.4	-9.6	-10.4	-9.4	-10.8	-10.2	-6.6	-6.8	-4.8	-4.6
19	-10.6	-9.4	-8.0	-8.6	-7.6	-7.2	-3.8	-5.2	-4.2	-2.2
20	-12.4	-12.6	-13.8	-11.9	-12.8	-8.0	-8.8	-7.8	-3.8	-3.0

TABLE 7 SHEAR FLEXIBILITY OF PARTITIONS IN TEST BUILDING

Shear Flexibility Component	Partition in 5-ft. bay		Partition in 10-ft. bay	
	Welded on both faces	Welded on one face	Welded on both faces	Welded on one face
$C_1 = \frac{2\alpha a(1+\nu)}{E\ b\ t}$	$= \frac{2x1.5x51(1+0.25)}{30000x48x0.025}$ $= 5.3125x10^{-3}$ in/kip	$= 5.3125x10^{-3}$ in/kip	$= \frac{2x1.5x51(1+0.25)}{30000x108x0.025}$ $= 2.3611x10^{-3}$ in/kip	$= 2.3611x10^{-3}$ in/kip
$C_2 = \frac{(b^3 + a^3)}{6\ EA\ b^2}$	$= \frac{(48^3 + 51^3)}{6x29000x0.28x48^2}$ $= 2.1669x10^{-3}$ in/kip	$= 2.1669x10^{-3}$ in/kip	$= \frac{(108^3 + 51^3)}{6x29000x0.28x108^2}$ $= 2.4502x10^{-3}$ in/kip	$= 2.4502x10^{-3}$ in/kip
$C_3 = \frac{144\ K\ h^3 \ell^2}{Et^3\ a\ b\ d}$	-----	$= \frac{144x0.073x1.5^3x3^2}{30000x0.025^3x48x51x6}$ $= 46.3769x10^{-3}$ in/kip	-----	$= \frac{144x0.073x1.5^3x3^2}{30000x0.025x48x108x6}$ $= 20.6152x10^{-3}$ in/kip
$C_4 = \frac{2\ s\ p}{b^2}\ (a+b)$	$= \frac{2x0.008x3}{48^2}(51+48)$ $= 2.065x10^{-3}$ in/kip	$= \frac{2x0.008x6}{48^2}(51+48)$ $= 4.13x10^{-3}$ in/kip	$= \frac{2x0.008x3}{108^2}(51+108)$ $= 0.65x10^{-3}$ in/kip	$= \frac{2x0.008x6}{108^2}(51+108)$ $= 1.30x10^{-3}$ in/kip
$C_5 = \frac{a\ p's'}{b\ b_o}$	$= \frac{51x4x0.08}{48x24}$ $= 14.24x10^{-3}$ in/kip	$= 14.24x10^{-3}$ in/kip	$= \frac{51x4x0.08}{108x36}$ $= 4.16x10^{-3}$ in/kip	$= 4.16x10^{-3}$ in/kip
$C = \sum_{1}^{5} C_i$	$23.78x10^{-3}$ in/kip	$72.23x10^{-3}$ in/kip	$9.62x10^{-3}$ in/kip	$30.89x10^{-3}$ in/kip

TABLE 8 REDUCTION IN DRIFT OF CLAD FRAMES

Test No.	Δ_1	Δ_2	Δ_3	$\Delta_2-\Delta_1$	Reduction in Drift
2c	20.2	62.2	89.4	42.0	-
3a	16.8	48.8	70.0	32.0	23.8%
3b	13.0	34.9	50.7	21.9	47.8%
3c	14.0	38.6	55.8	24.6	41.8%
3d	10.0	22.7	33.2	12.7	67.3%
3e	9.1	19.0	28.2	9.9	76.3%

TABLE 9 STORY SHEAR IN TEST BUILDING

Test No.	First Story Shear		Second Story Shear		Third Story Shear	
	Measured	Theoretical	Measured	Theoretical	Measured	Theoretical
1	0.965	1.000	0.988	1.000	0.977	1.000
2a	0.955	1.000	0.986	1.000	0.973	1.000
2b	0.932	1.000	0.978	1.000	0.973	1.000
2c	0.940	1.000	0.986	1.000	0.966	1.000
2d	0.950	1.000	0.986	1.000	0.998	1.000
3a	0.933	1.000	0.605	0.555	0.962	1.000
3b	0.938	1.000	0.174	0.090	0.972	1.000
3c	0.952	1.000	0.314	0.210	1.000	1.000
3d	0.938	1.000	-0.221	-0.320	0.962	1.000
3e	0.934	1.000	-0.326	-0.440	0.955	1.000

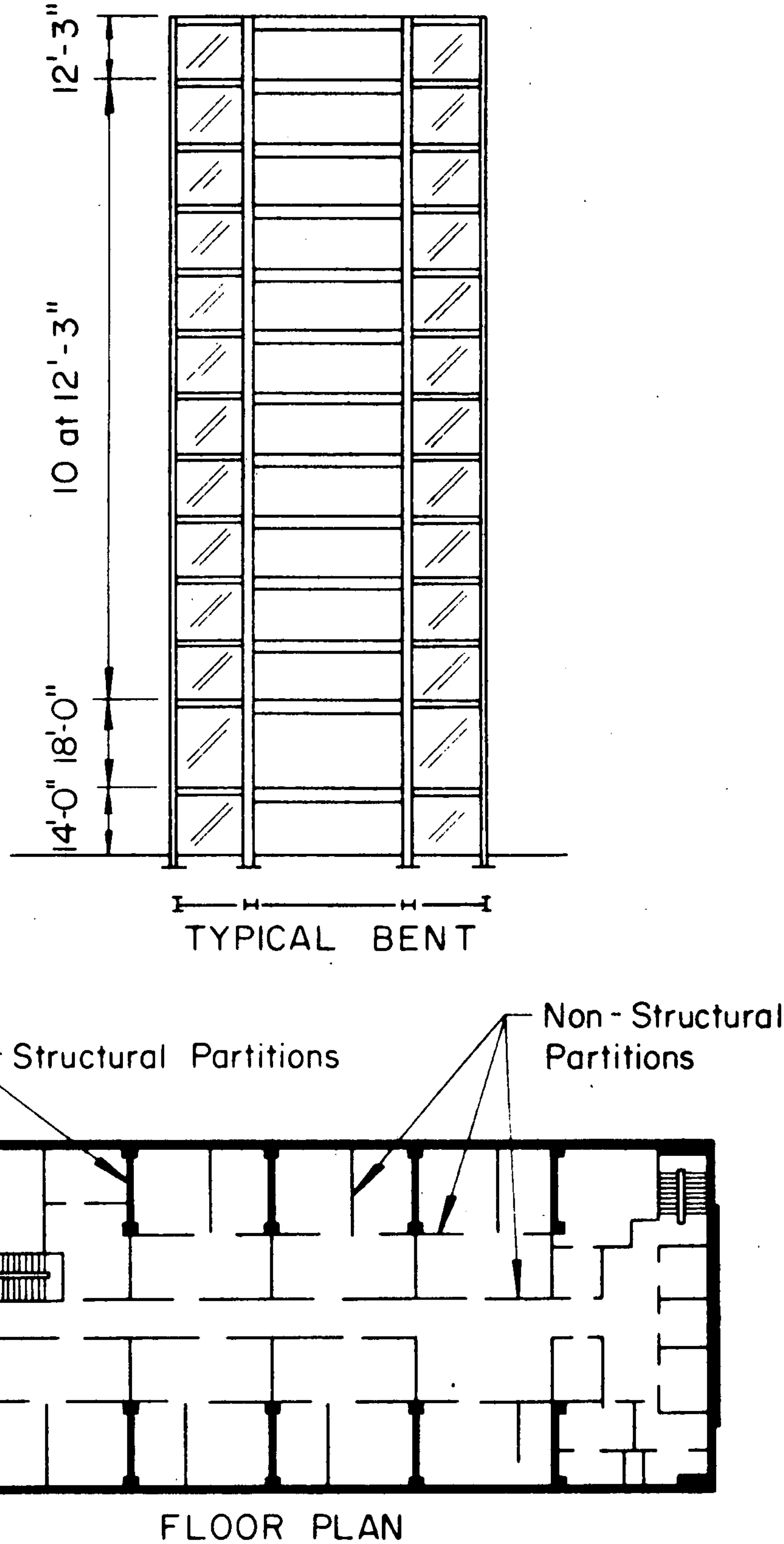


FIG. 1 STRUCTURAL PARTITIONS

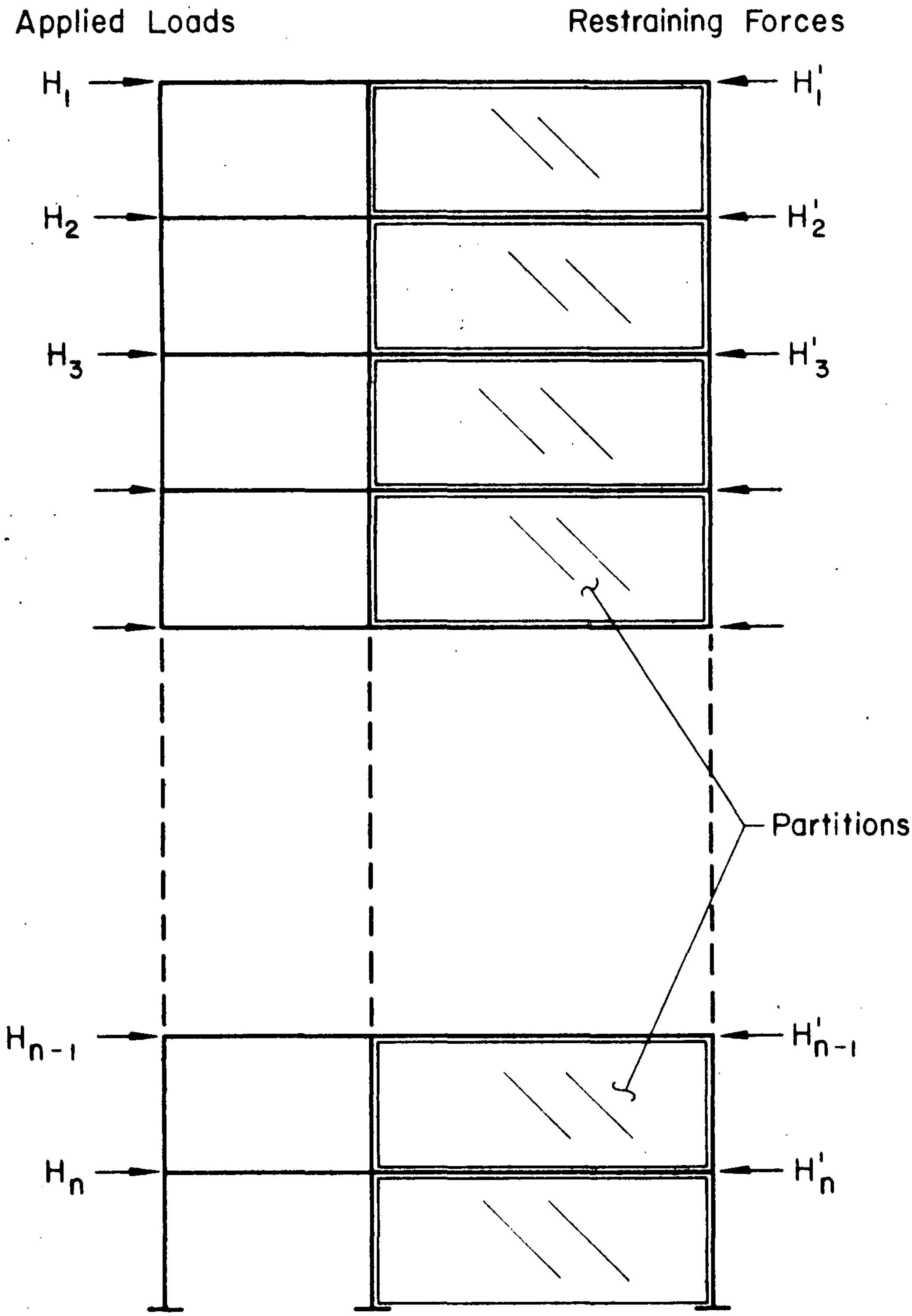


FIG. 2 NET FORCES ON FRAME: H - H'

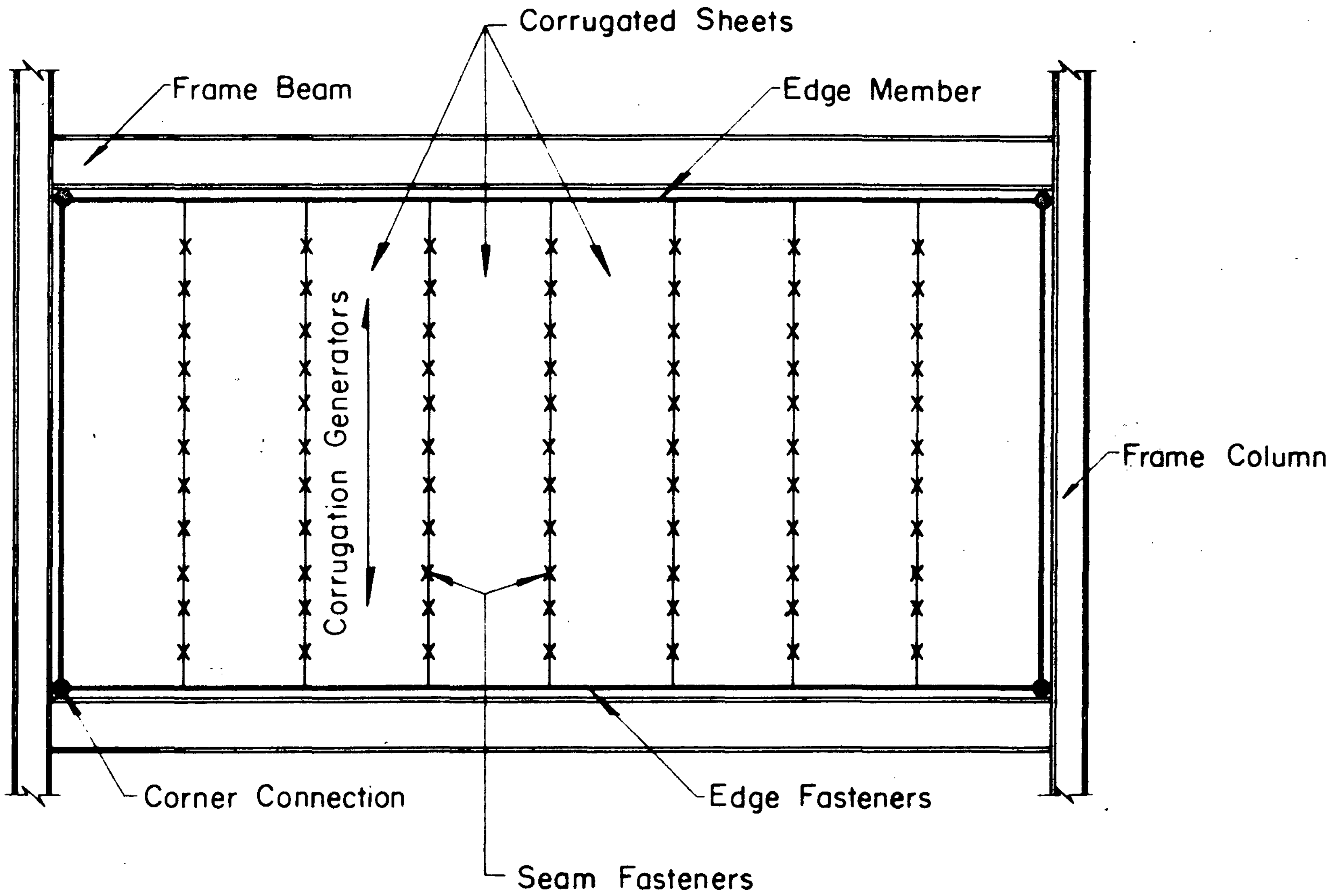


FIG. 3 TYPICAL STRUCTURAL PARTITION

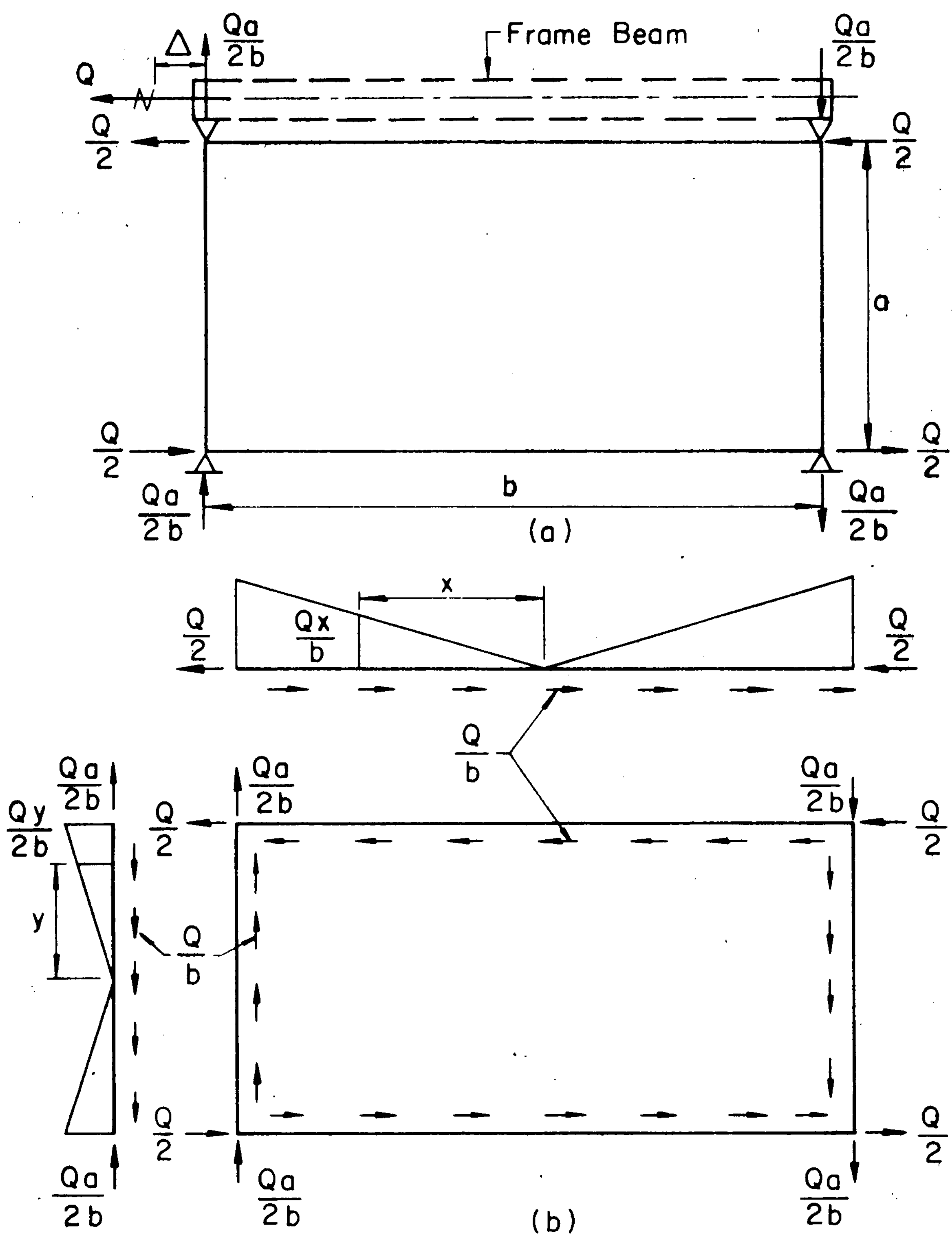
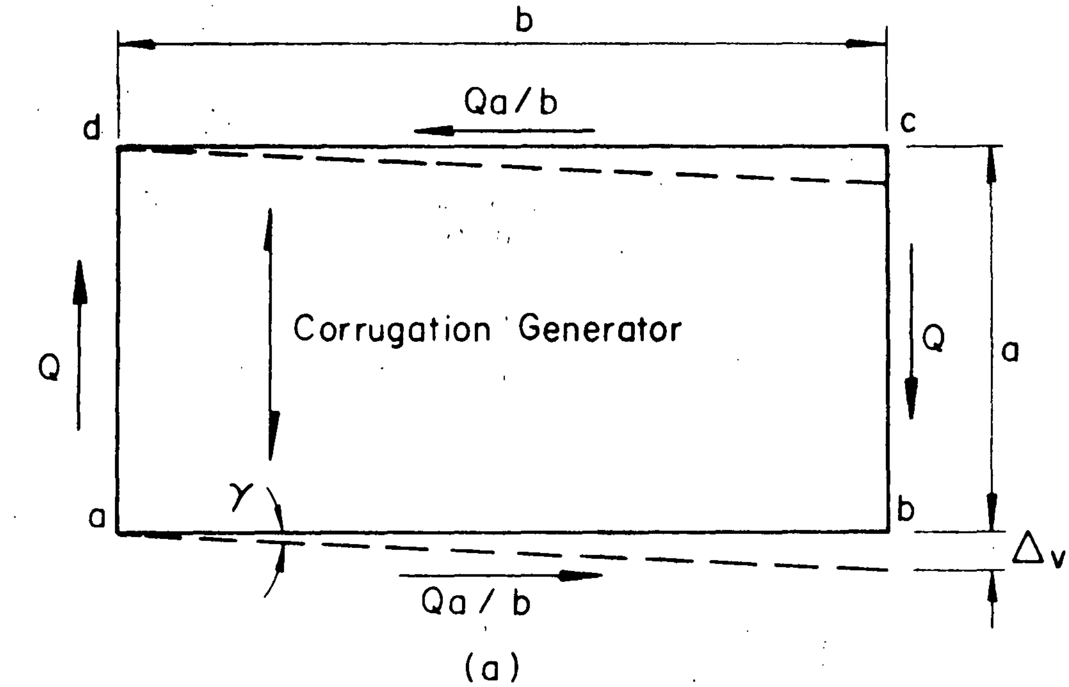


FIG. 4 FORCES ON A STRUCTURAL PARTITION



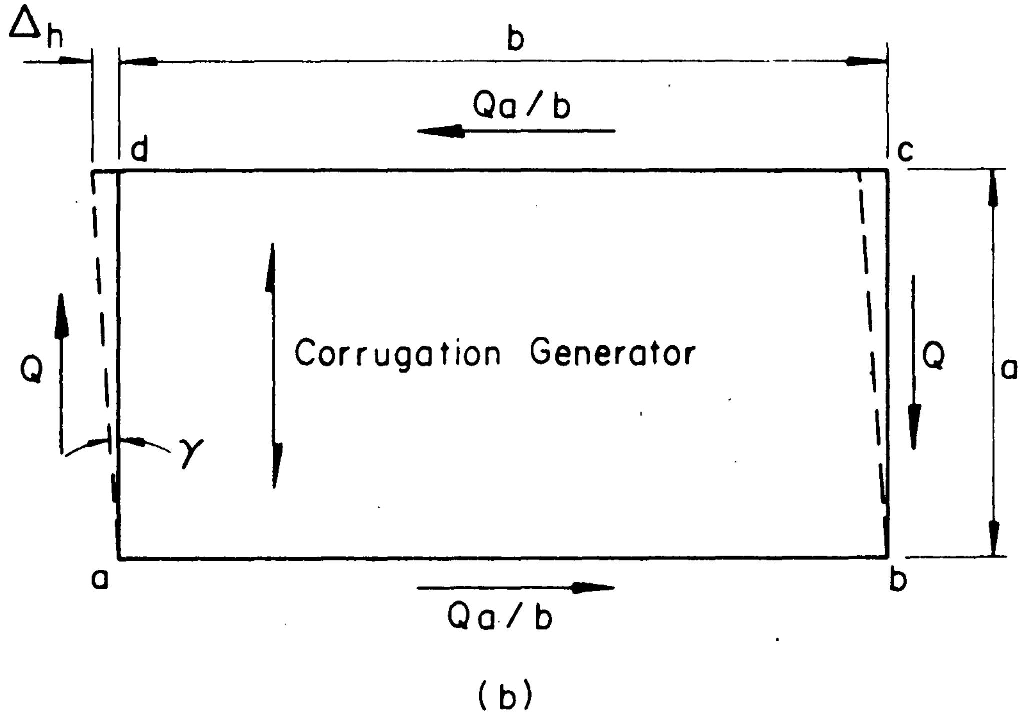


FIG. 5 FLEXIBILITY DUE TO BENDING OF CORRUGATION

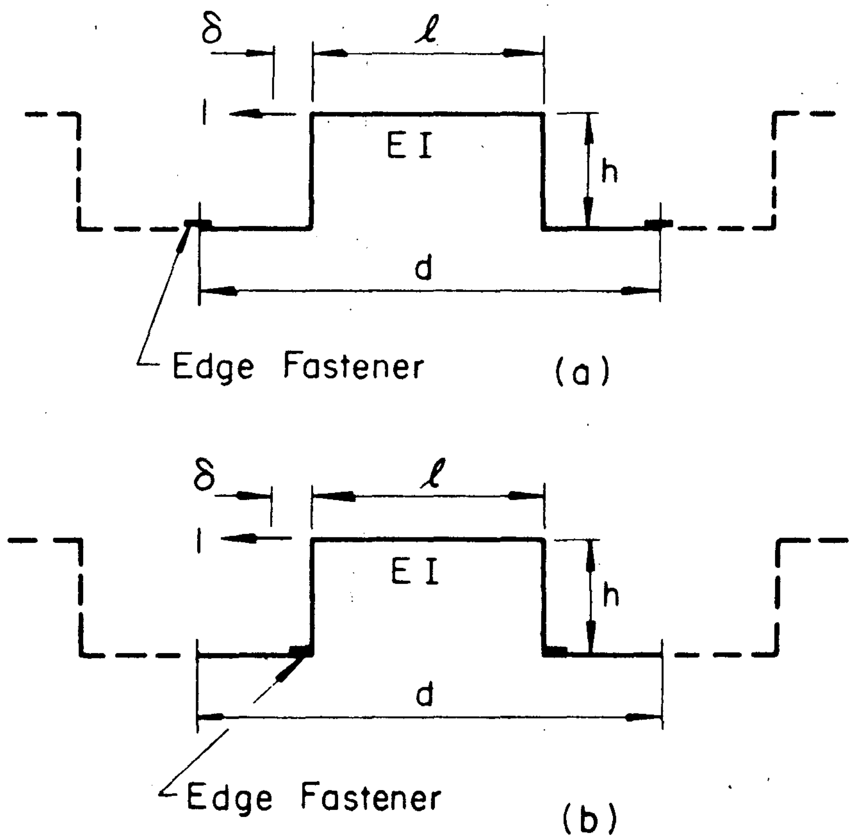


FIG. 6 BENDING OF CORRUGATION PROFILE

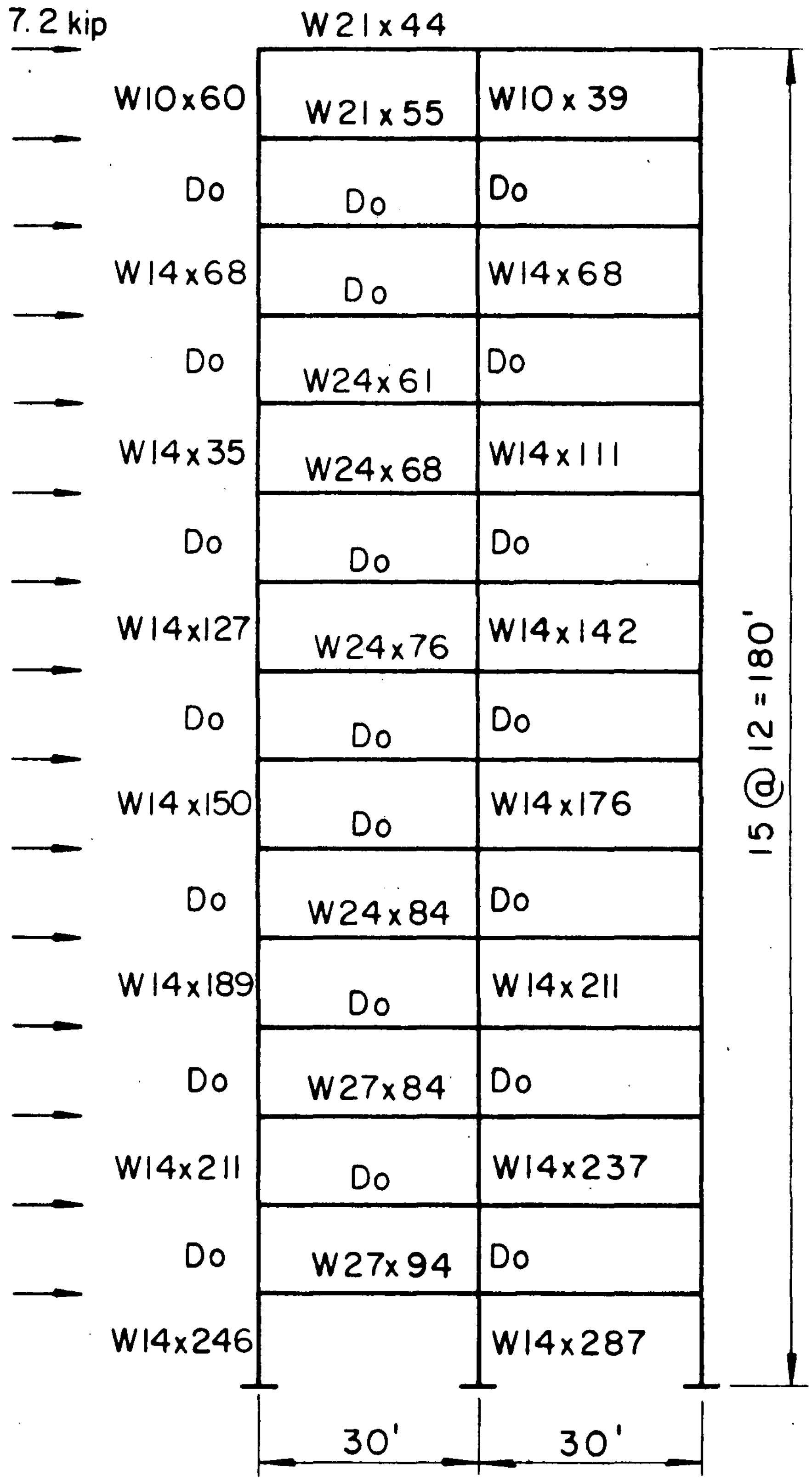


FIG. 7 DESCRIPTION OF DESIGN FRAME

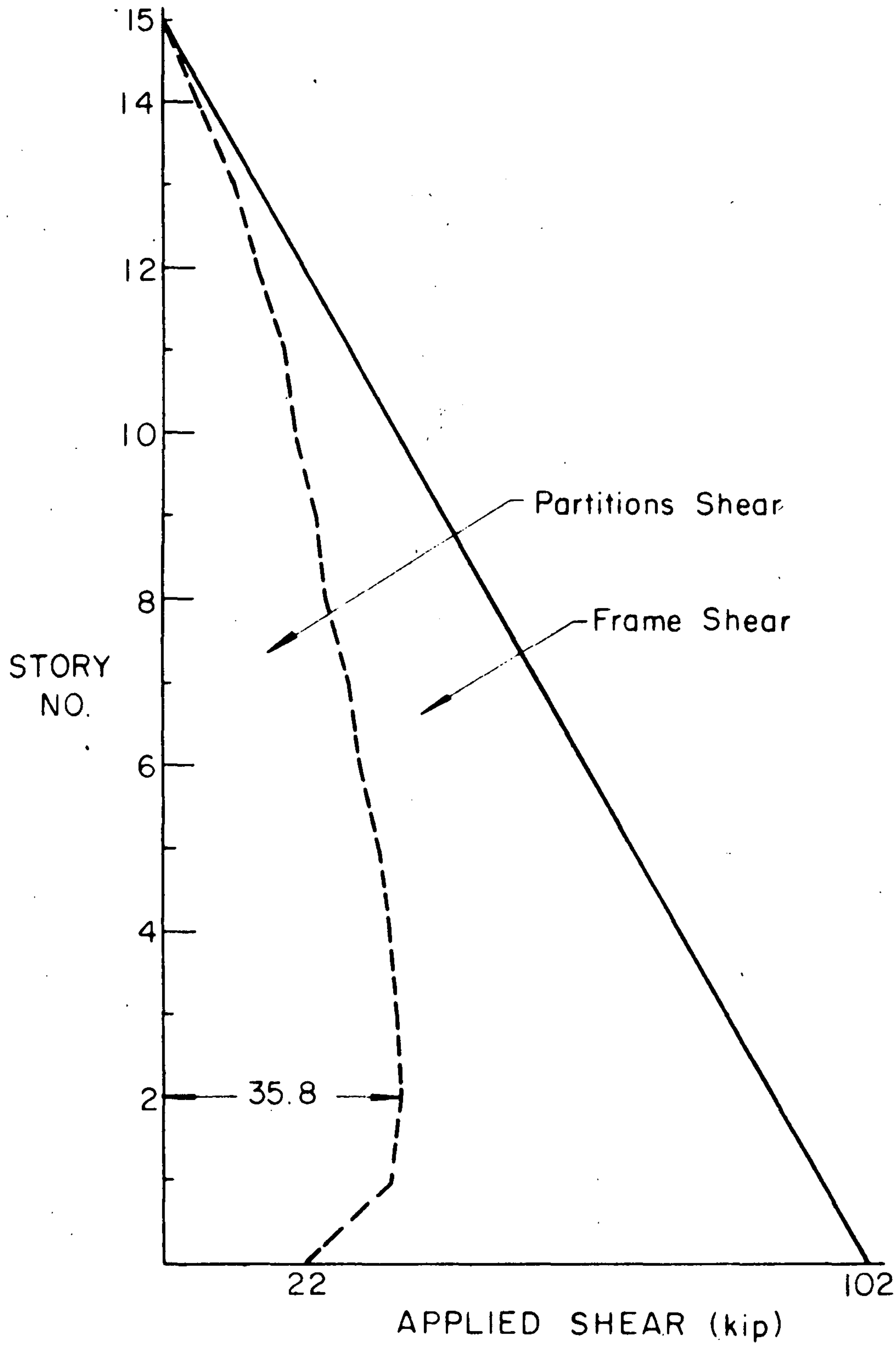


FIG. 8 SHEAR DISTRIBUTION - C = 7.91×10^{-3} in/kip

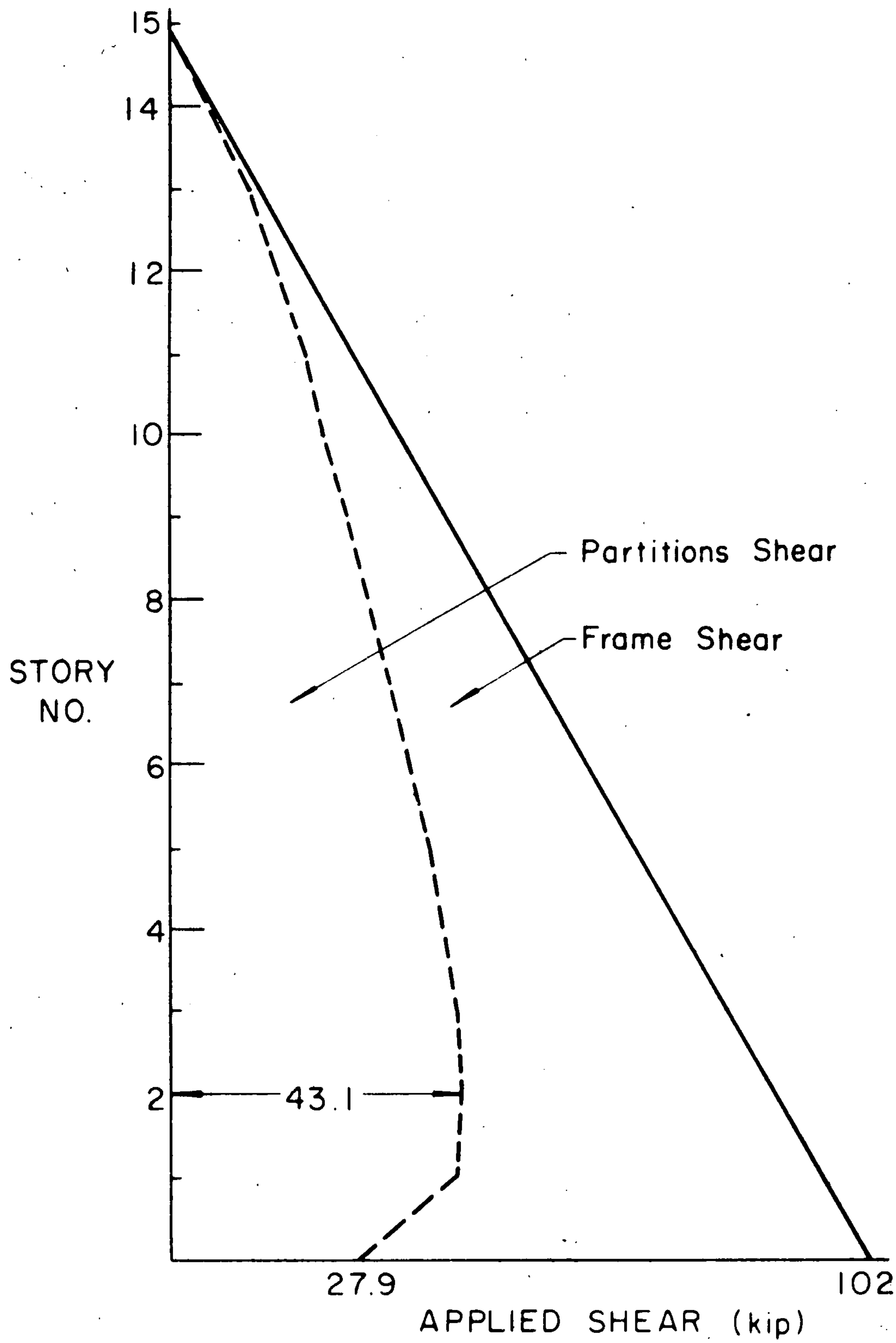


FIG. 9 SHEAR DISTRIBUTION - C = 5.76×10^{-3} in/kip

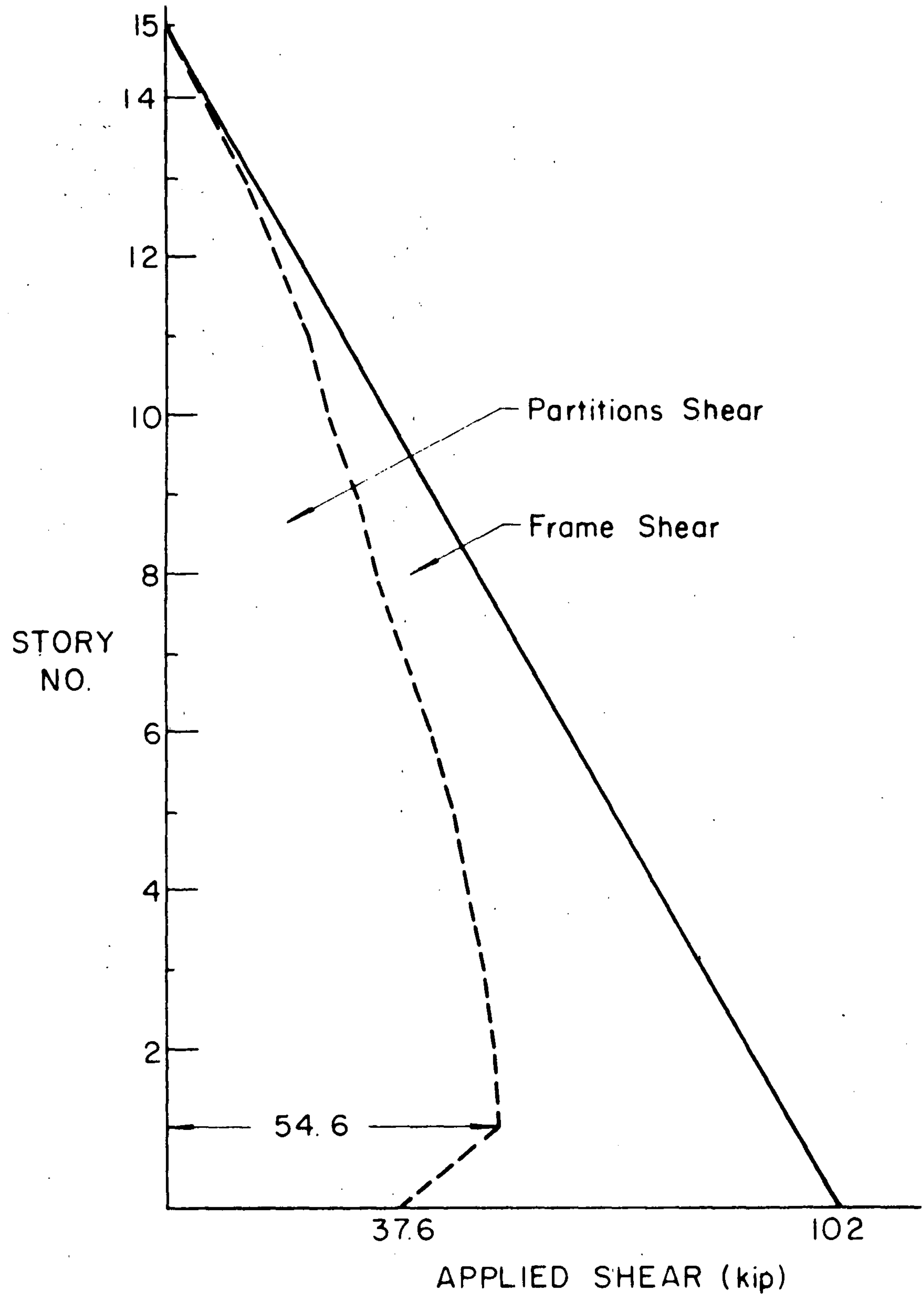


FIG. 10 SHEAR DISTRIBUTION - $C = 3.67 \times 10^{-3}$ in/kip

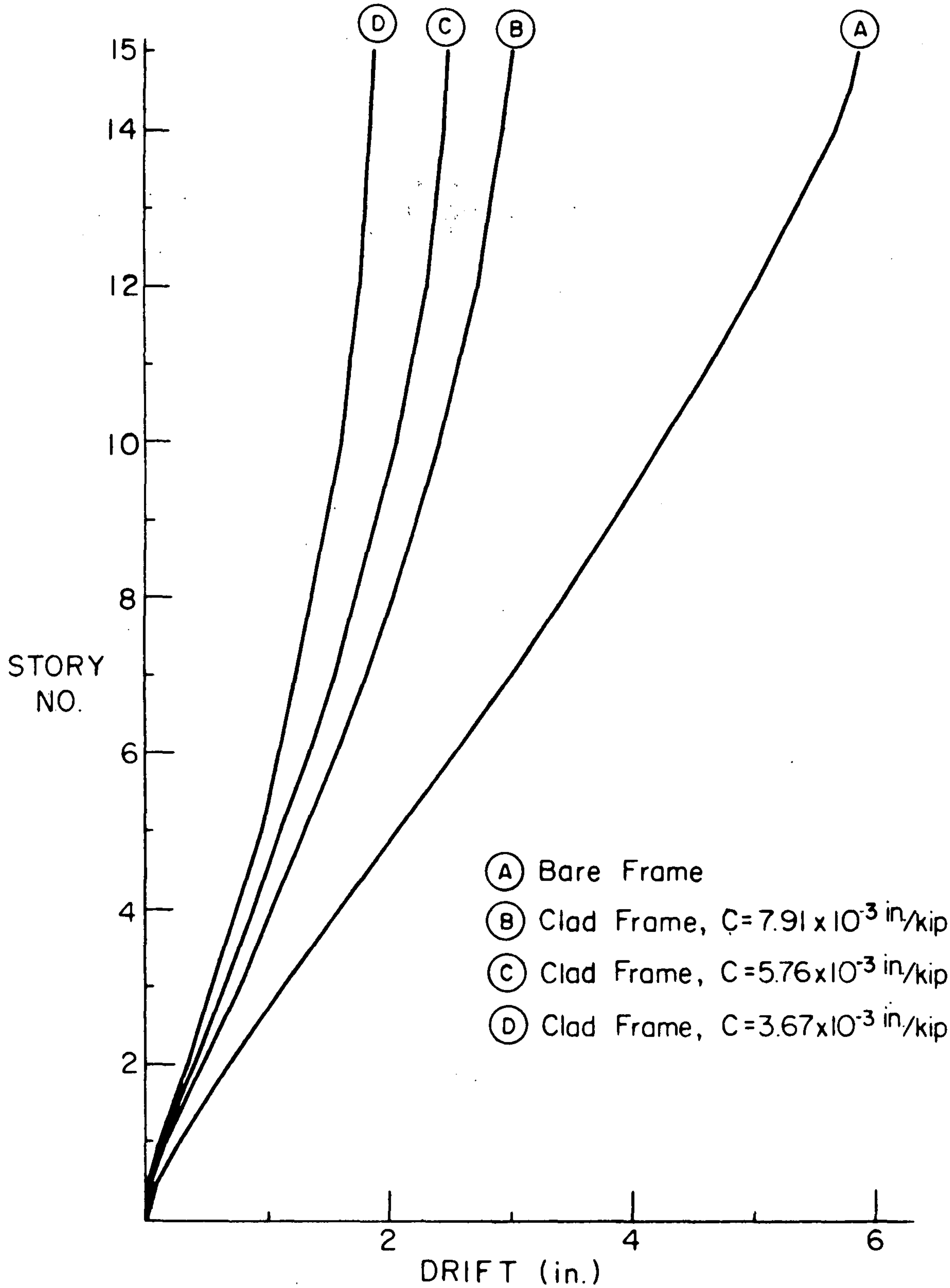


FIG. 11 DEFLECTED SHAPES OF DESIGN FRAME

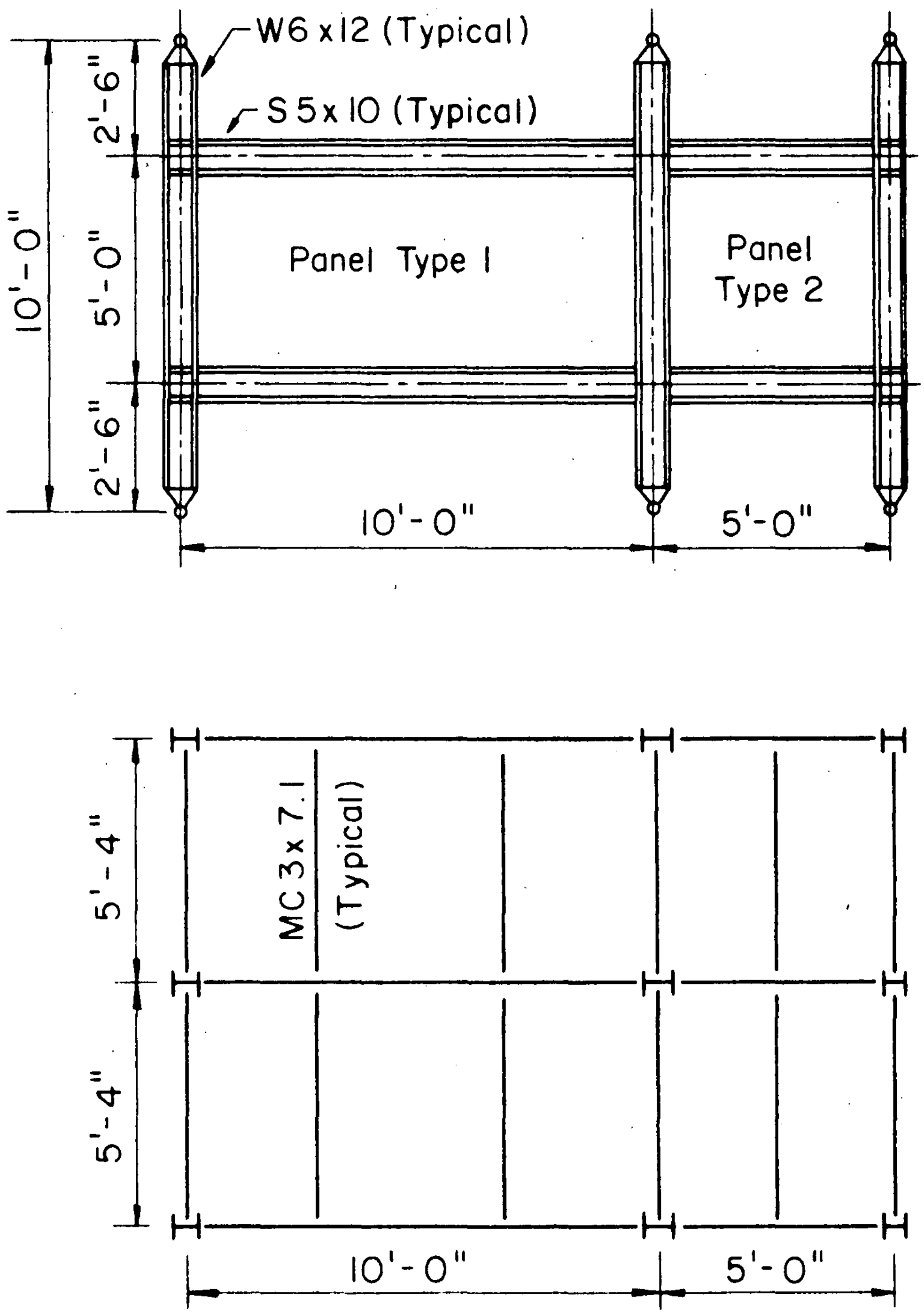


FIG. 12 STEEL FRAMING OF TEST BUILDING

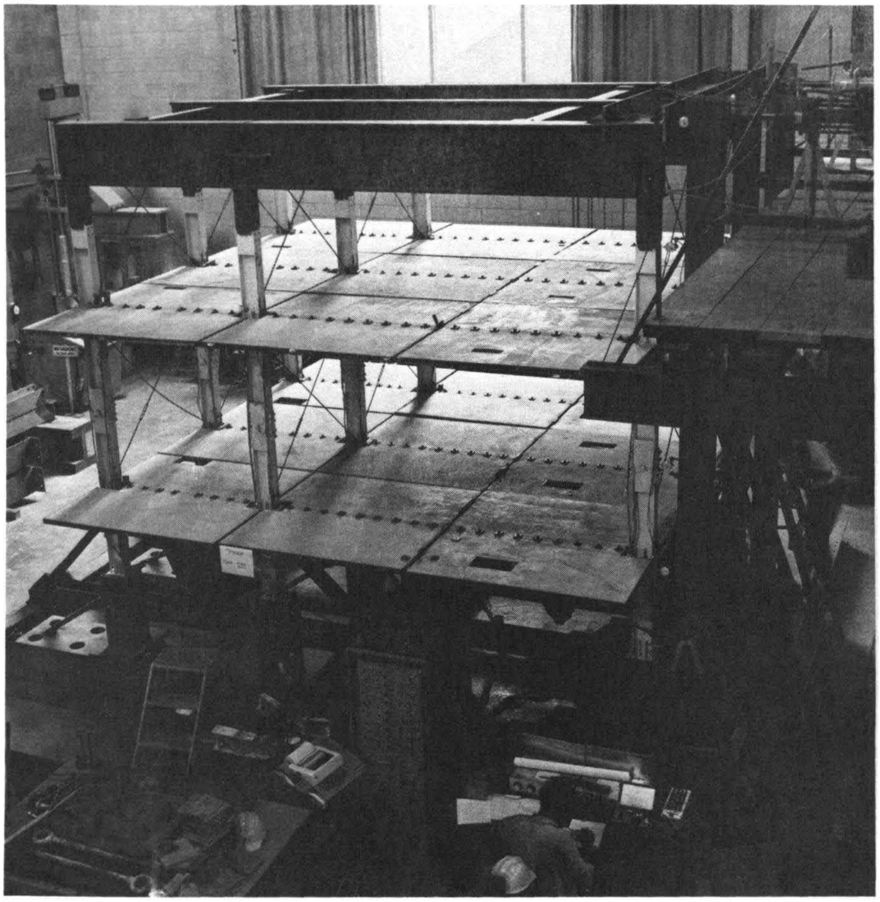


FIG. 13 TEST BUILDING WITH CONCRETE FLOORS

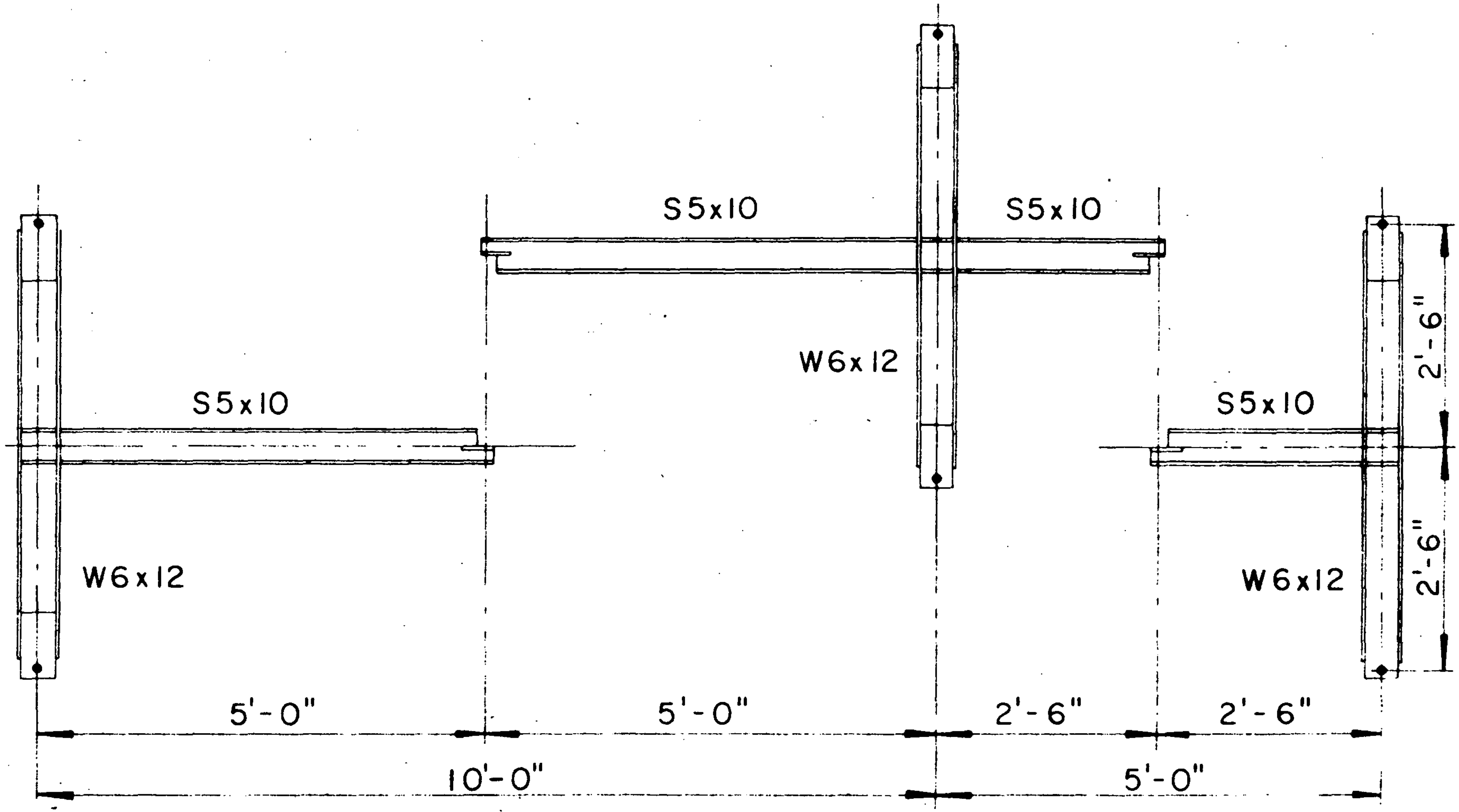


FIG. 14 TYPICAL ONE STORY SUBASSEMBLAGES

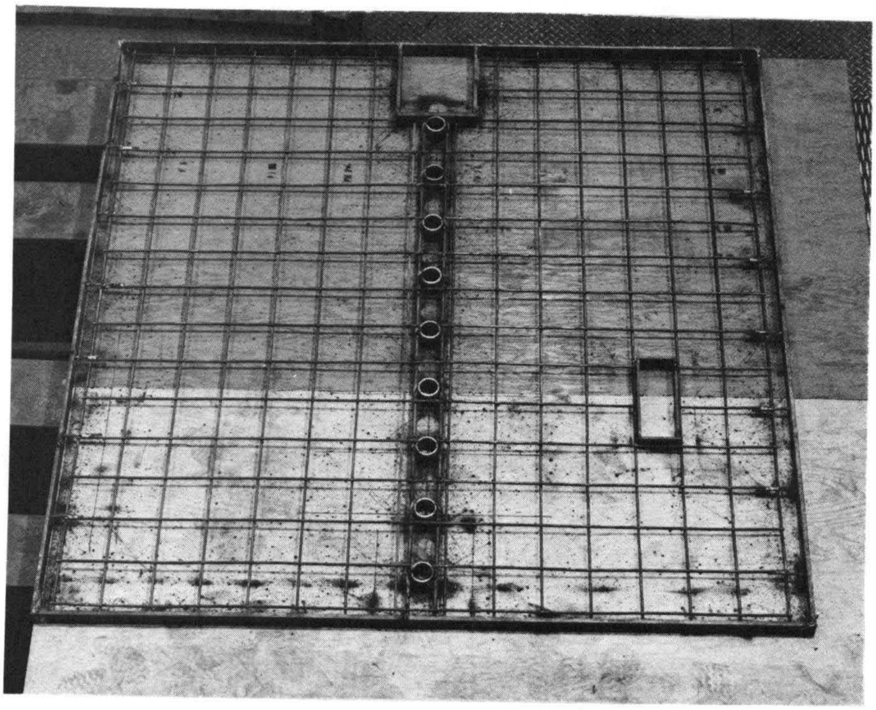


FIG. 15 UNFINISHED FLOOR PANEL

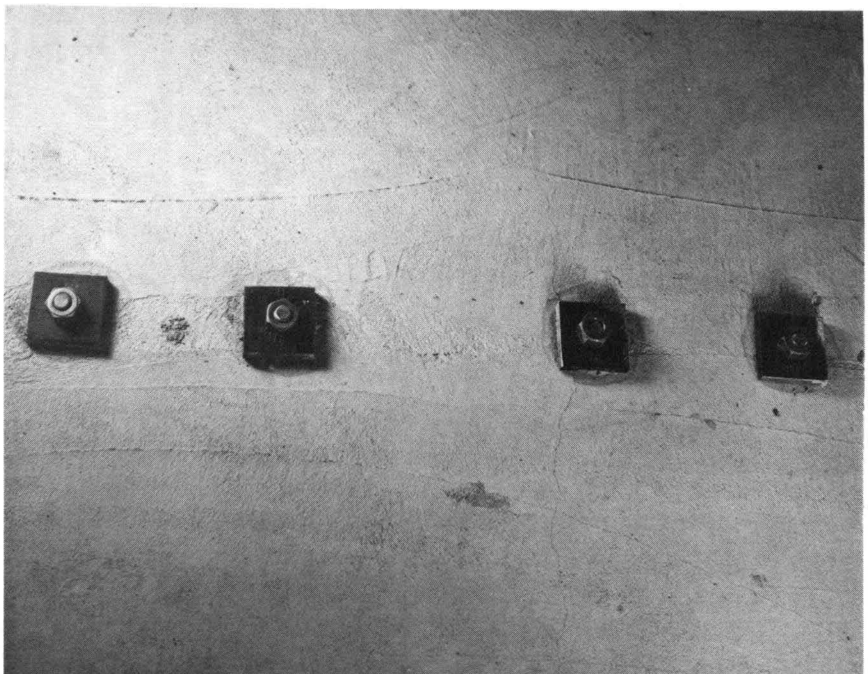


FIG. 16 FLOOR SHEAR CONNECTORS

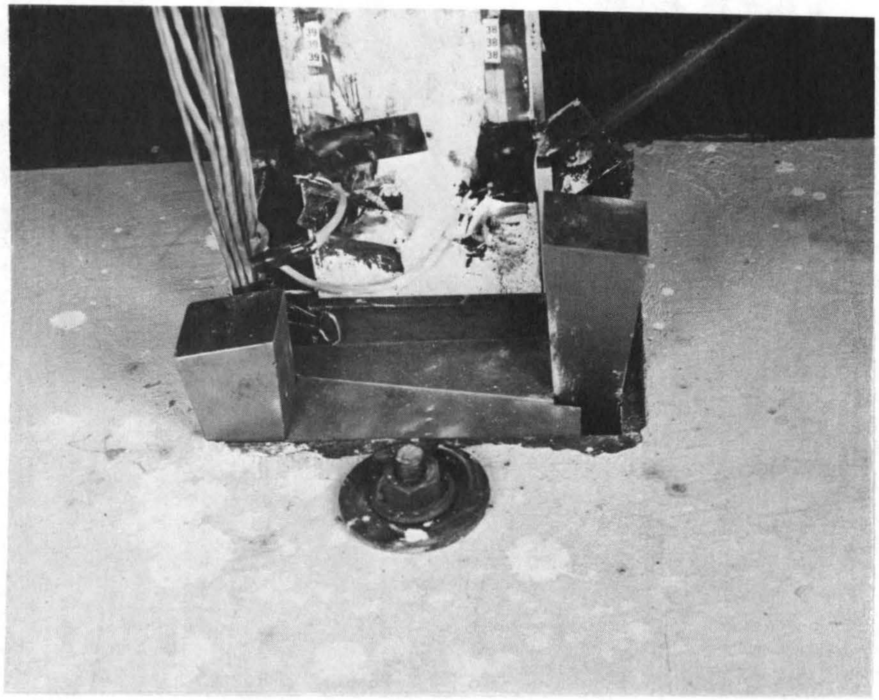


FIG. 17 WEDGE ASSEMBLY

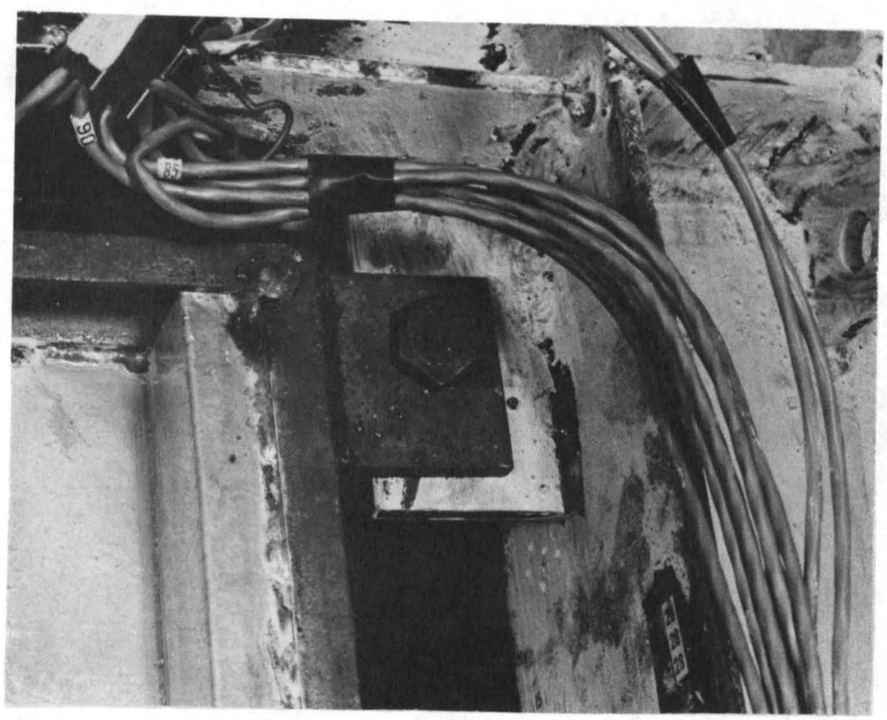


FIG. 18 TYPICAL PARTITION CORNER CONNECTION

FIG. 19 PARTITION LOWER EDGE CONNECTION

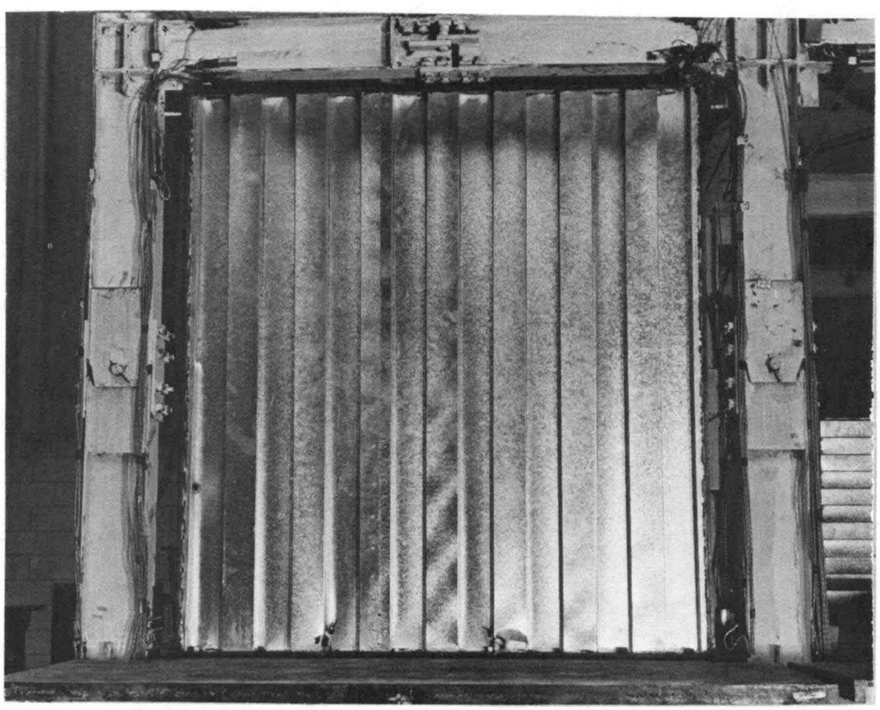


FIG. 20 5-ft. BAY PARTITION

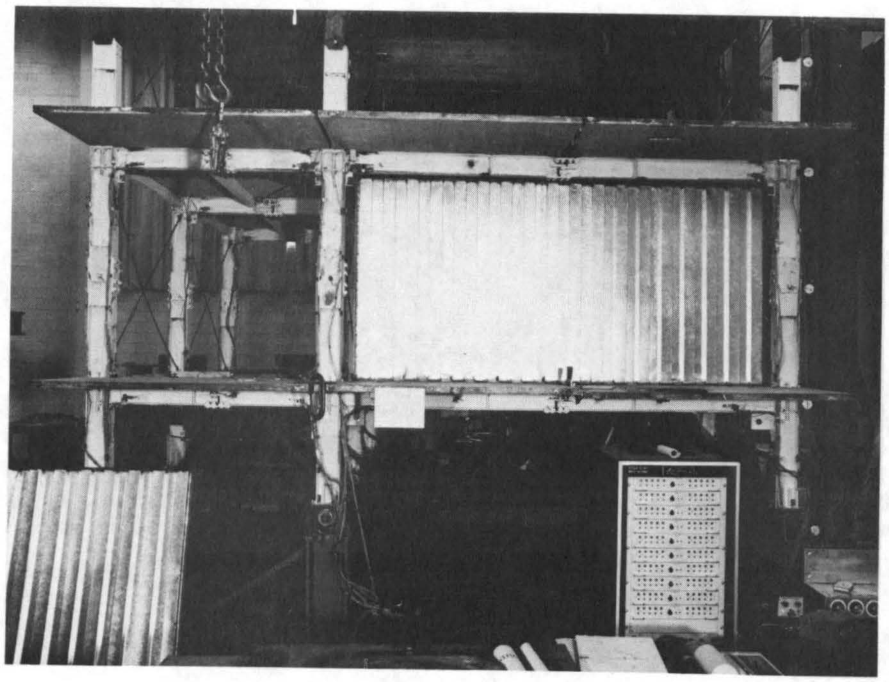


FIG. 21 TEST BUILDING WITH PARTITIONS IN 10-ft. BAY

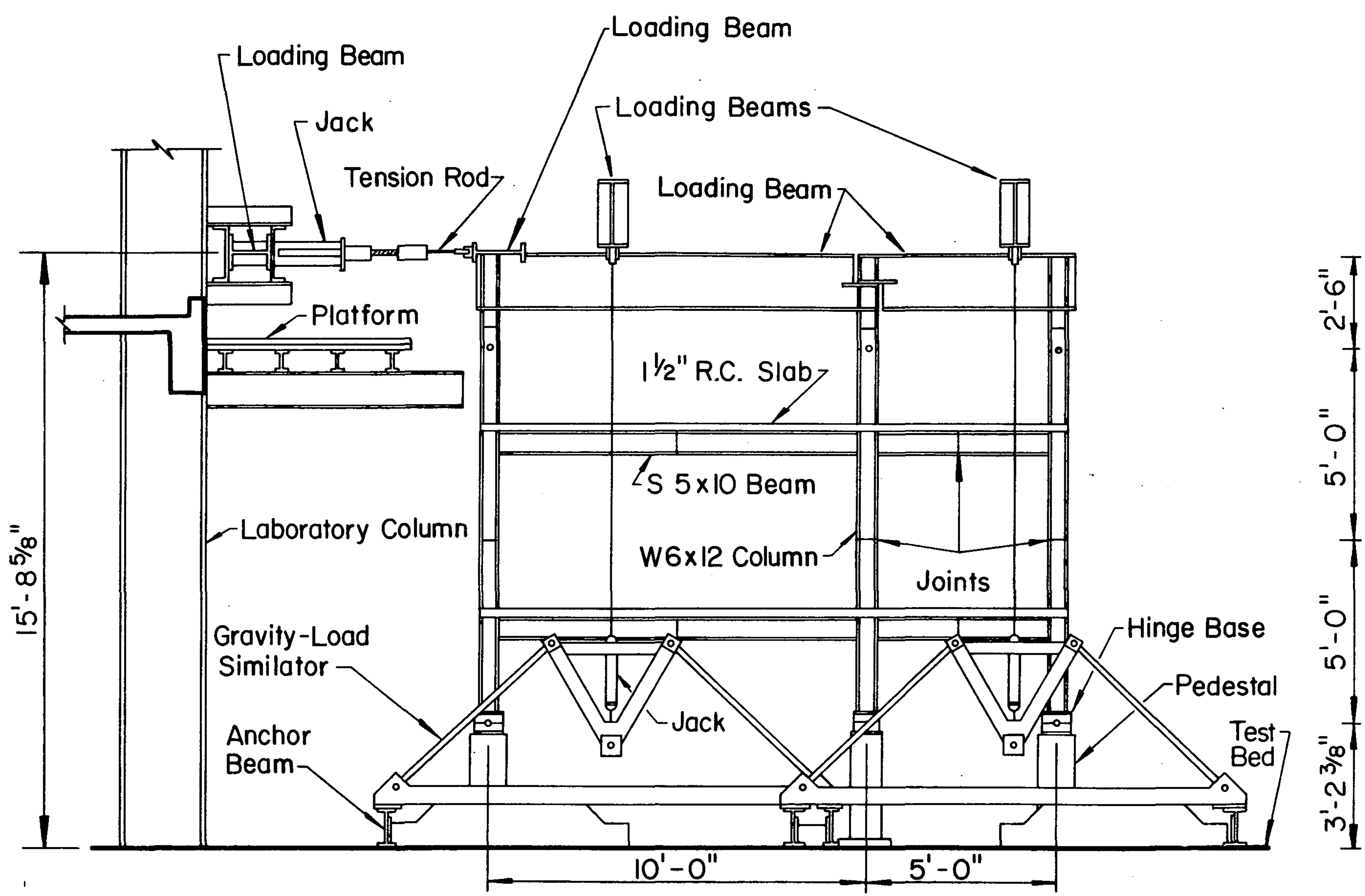


FIG. 22 ELEVATION VIEW OF TEST SETUP

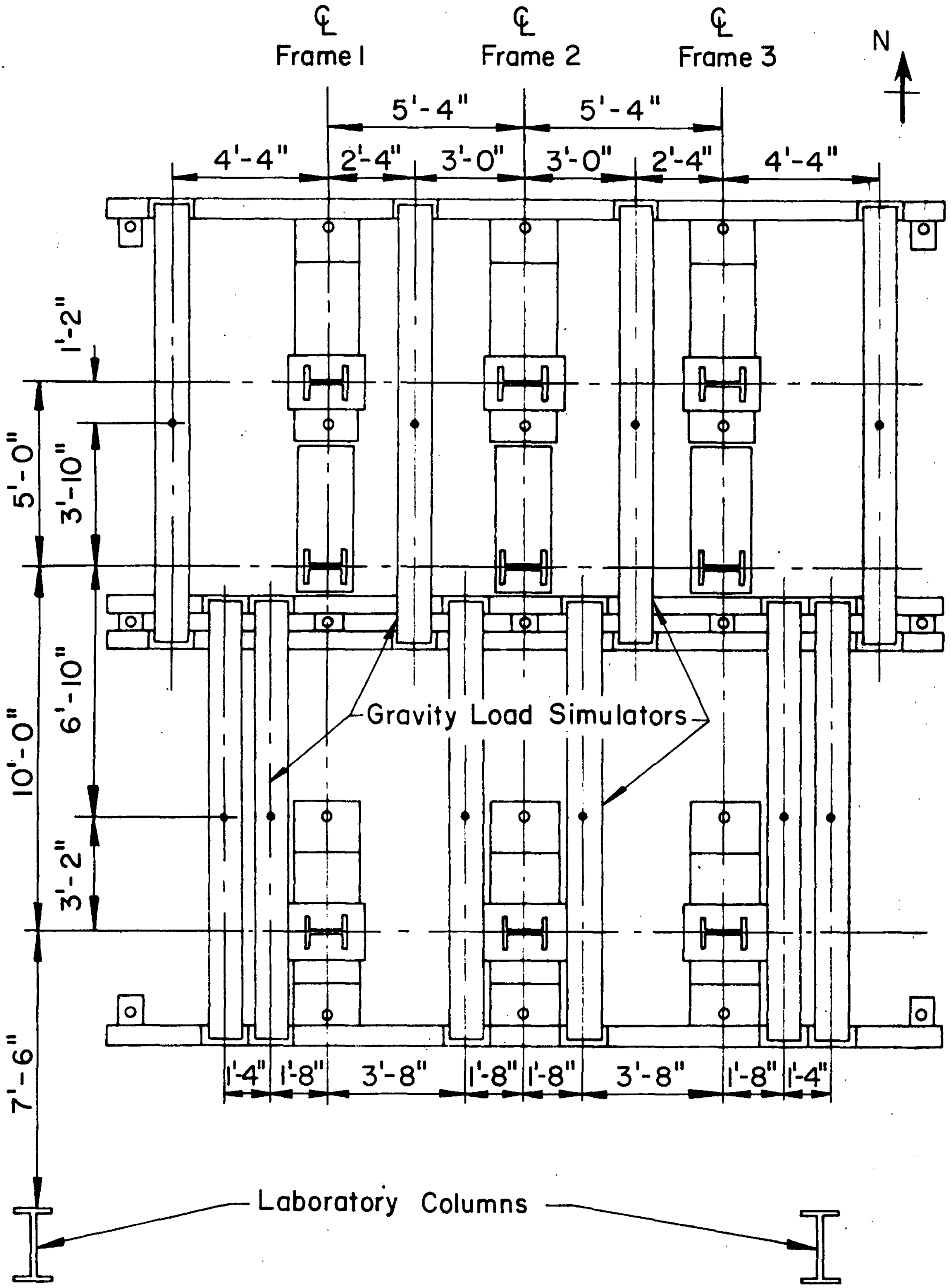


FIG. 23 PLAN OF TEST BED

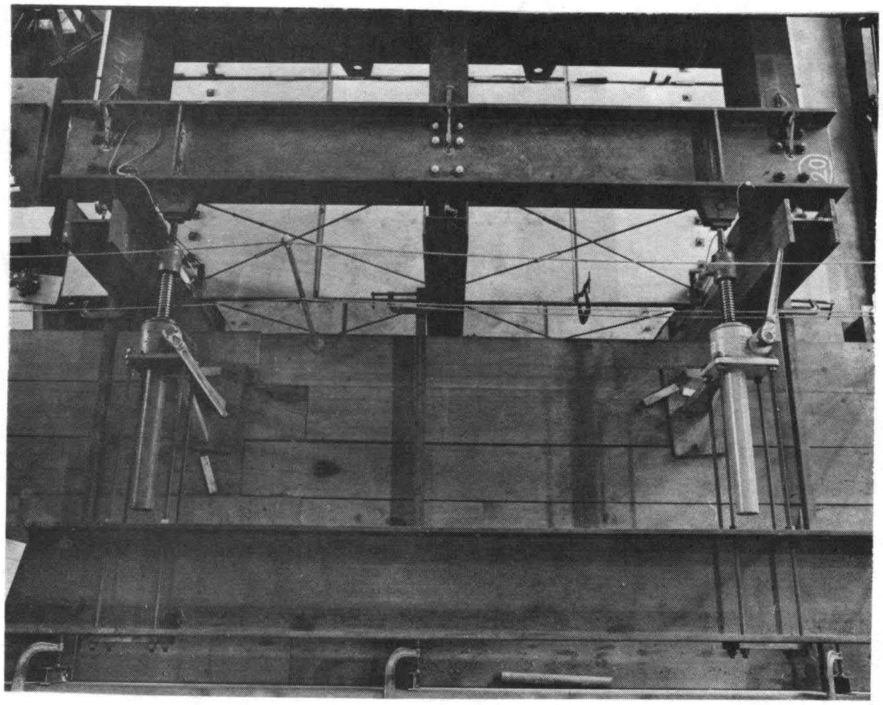


FIG. 24 HORIZONTAL LOADING APPARATUS

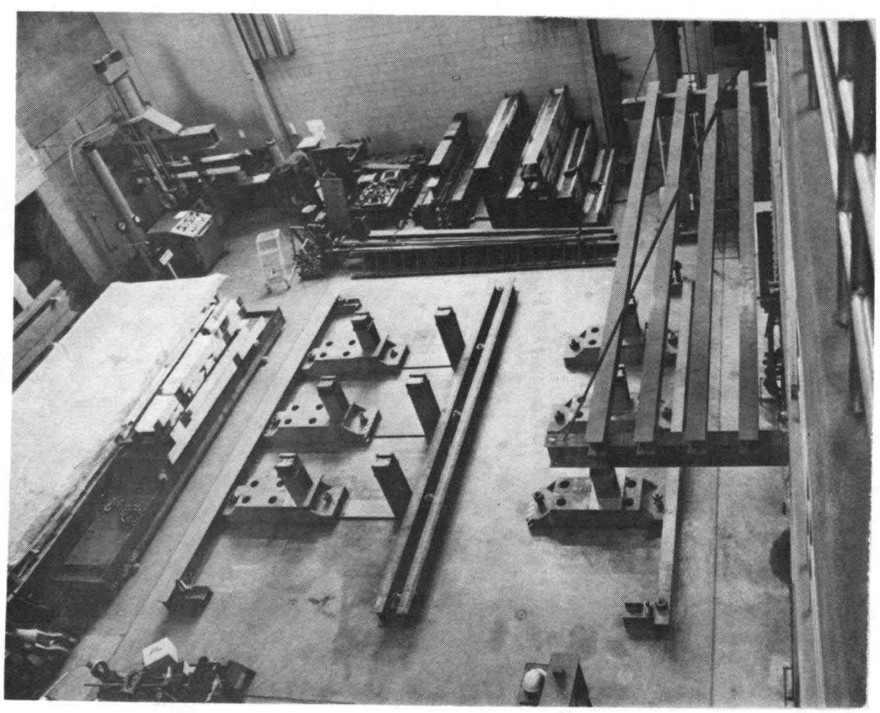


FIG. 25 TEST BED DURING ERECTION

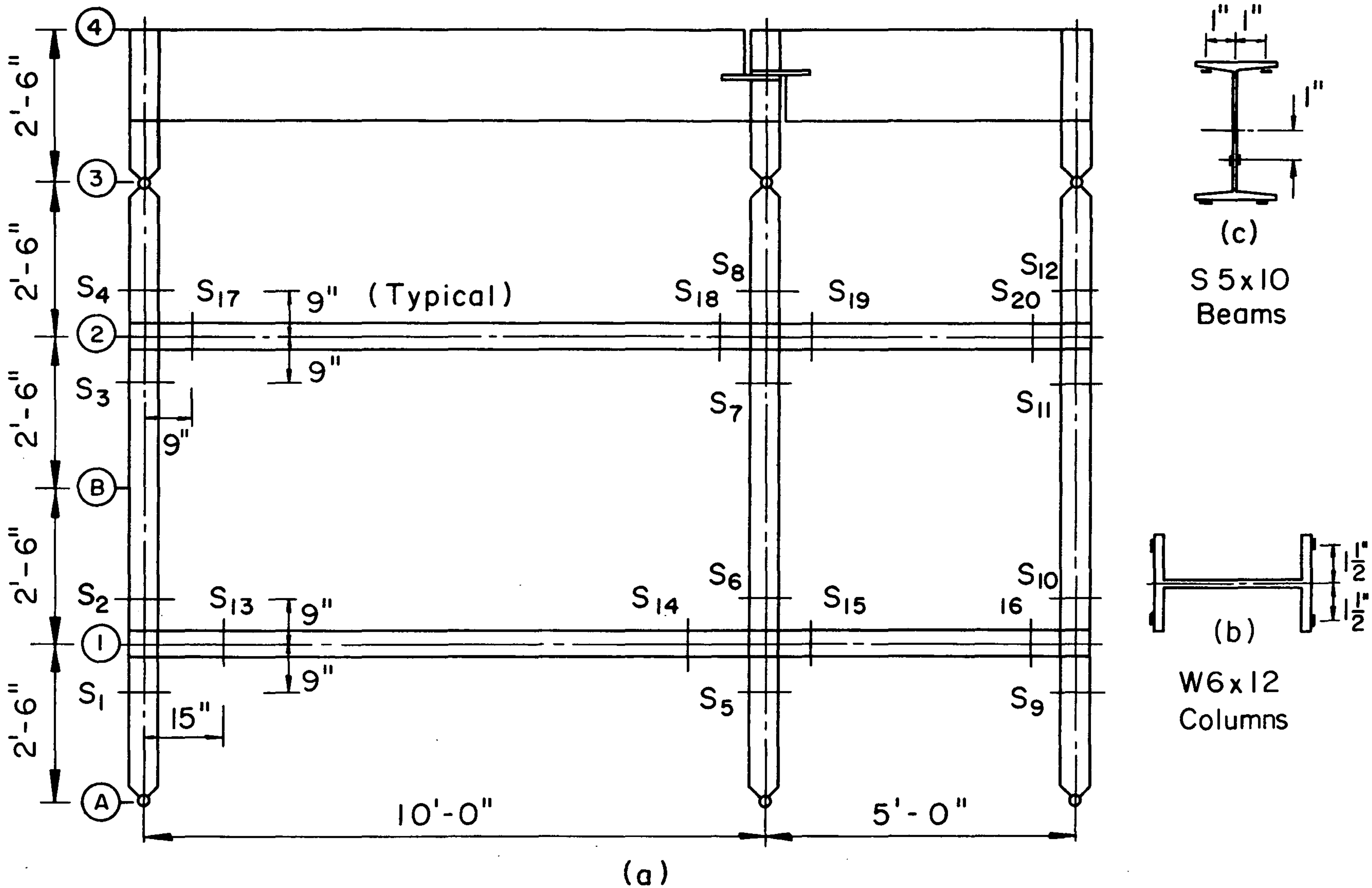


FIG. 26 INSTRUMENTATION

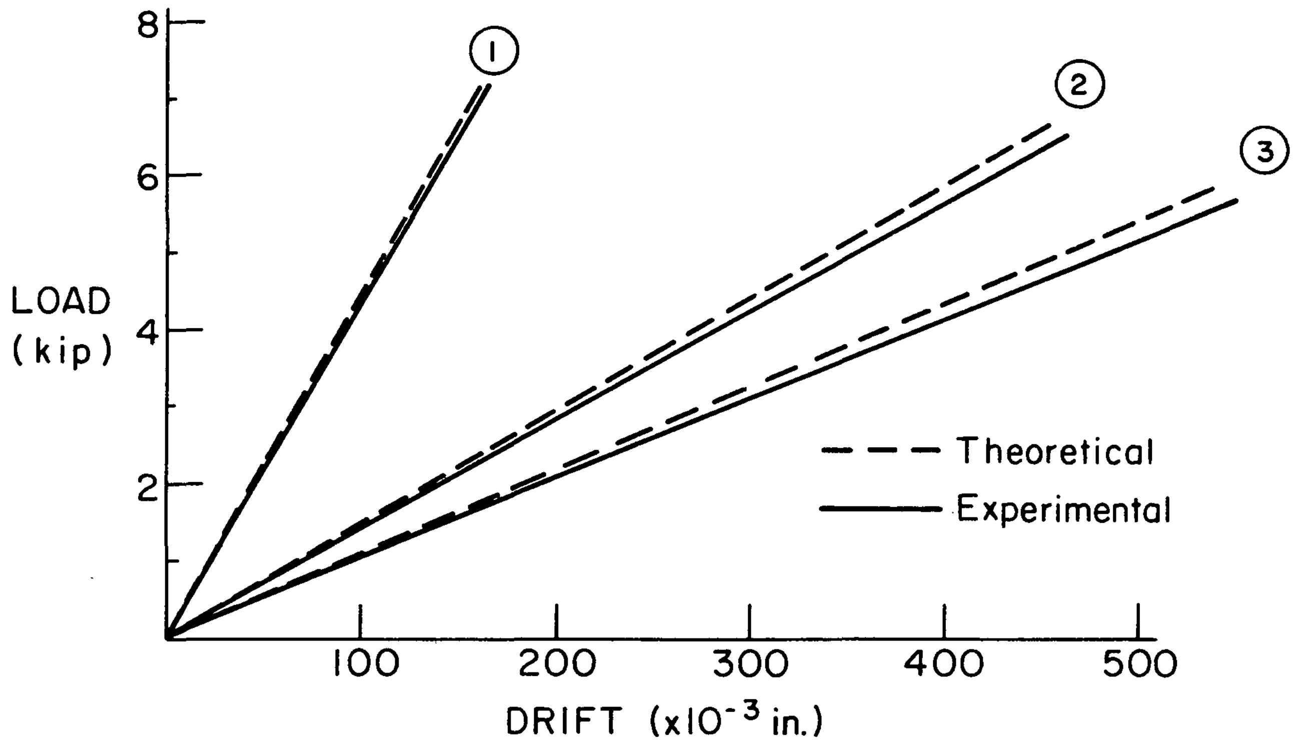


FIG. 27 DEFLECTIONS - TEST 1

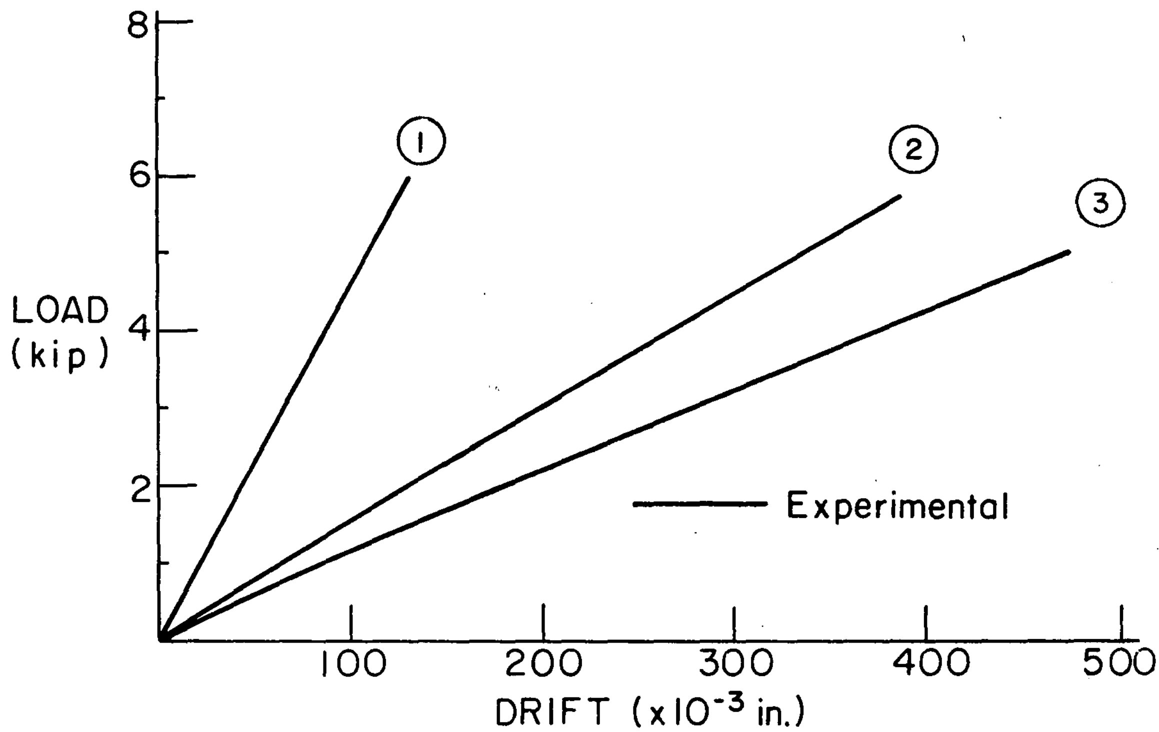


FIG. 28 DEFLECTIONS - TEST 2a

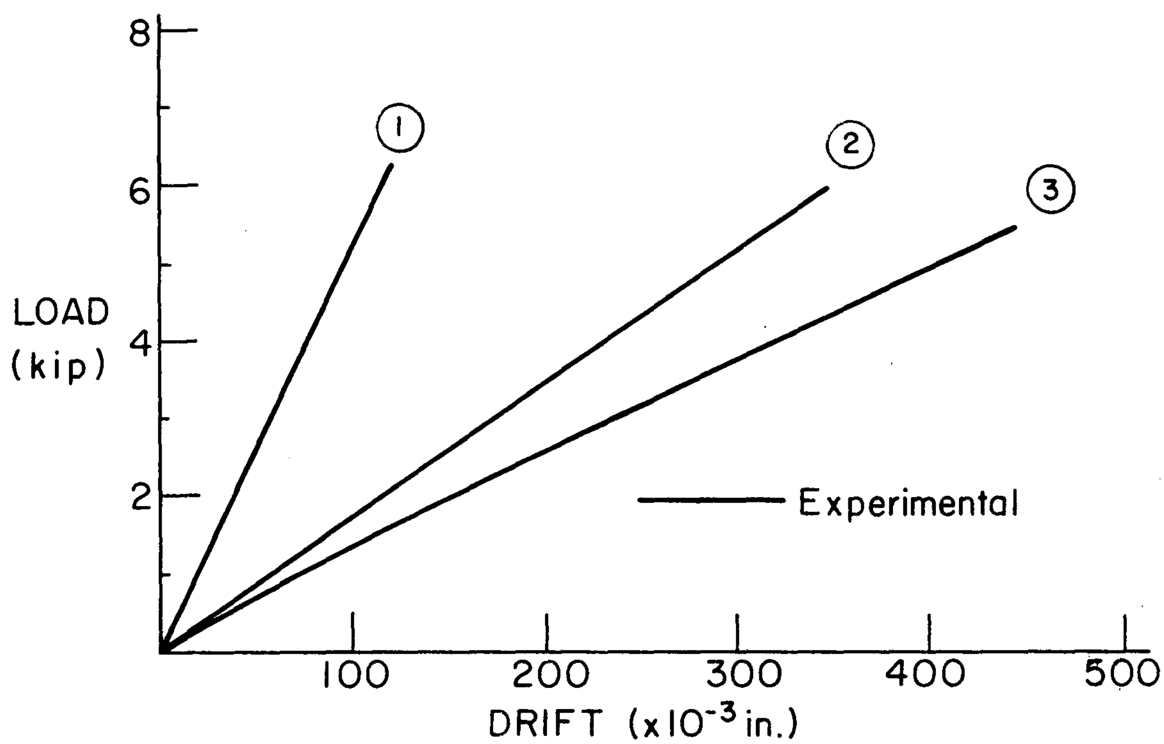


FIG. 29 DEFLECTIONS - TEST 2b

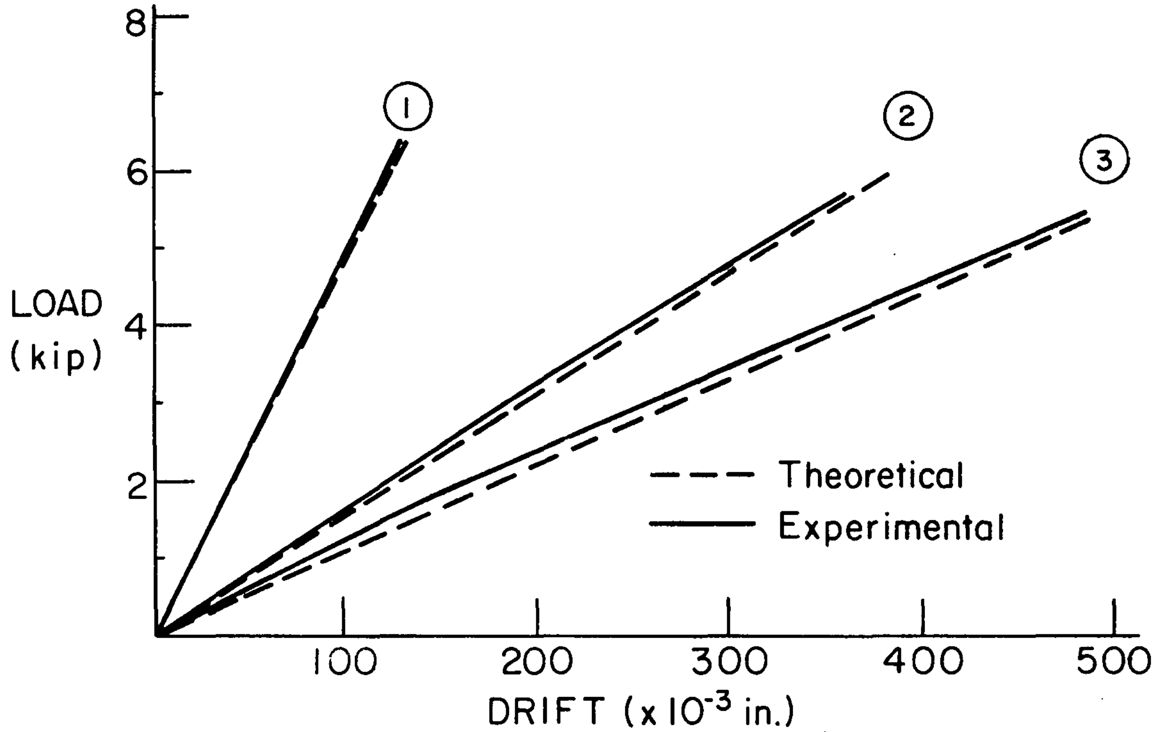


FIG. 30 DEFLECTIONS - TEST 2c

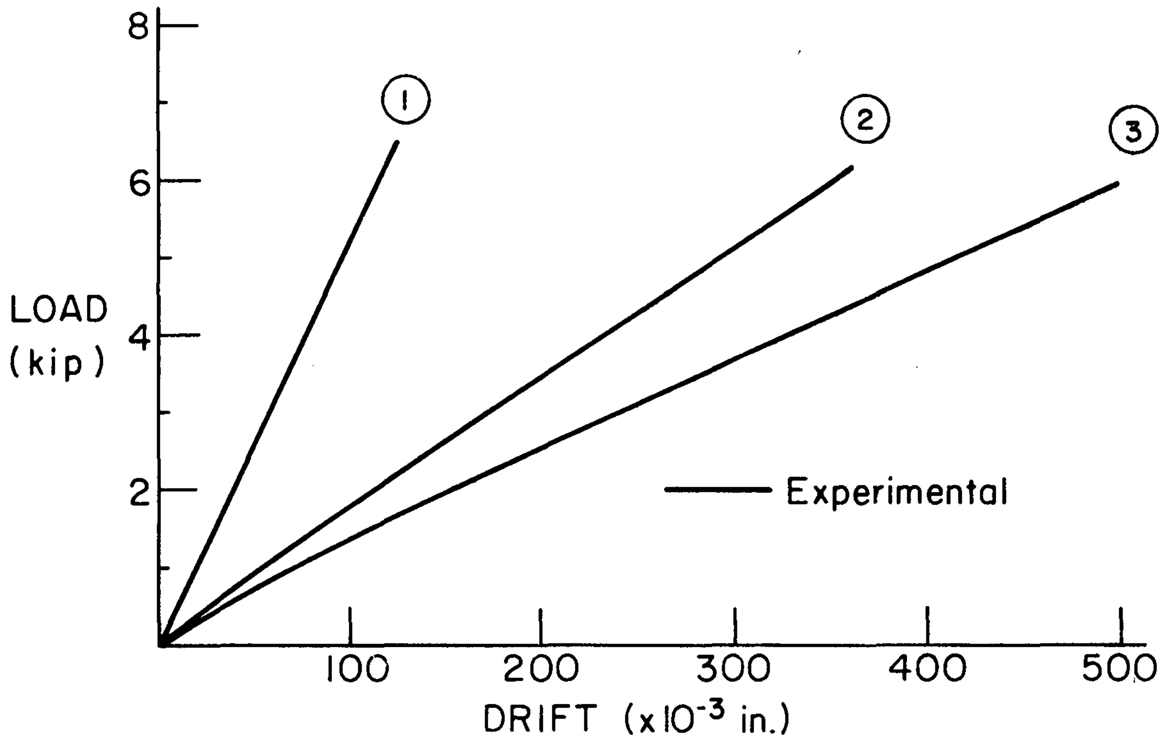


FIG. 31 DEFLECTIONS - TEST 2d

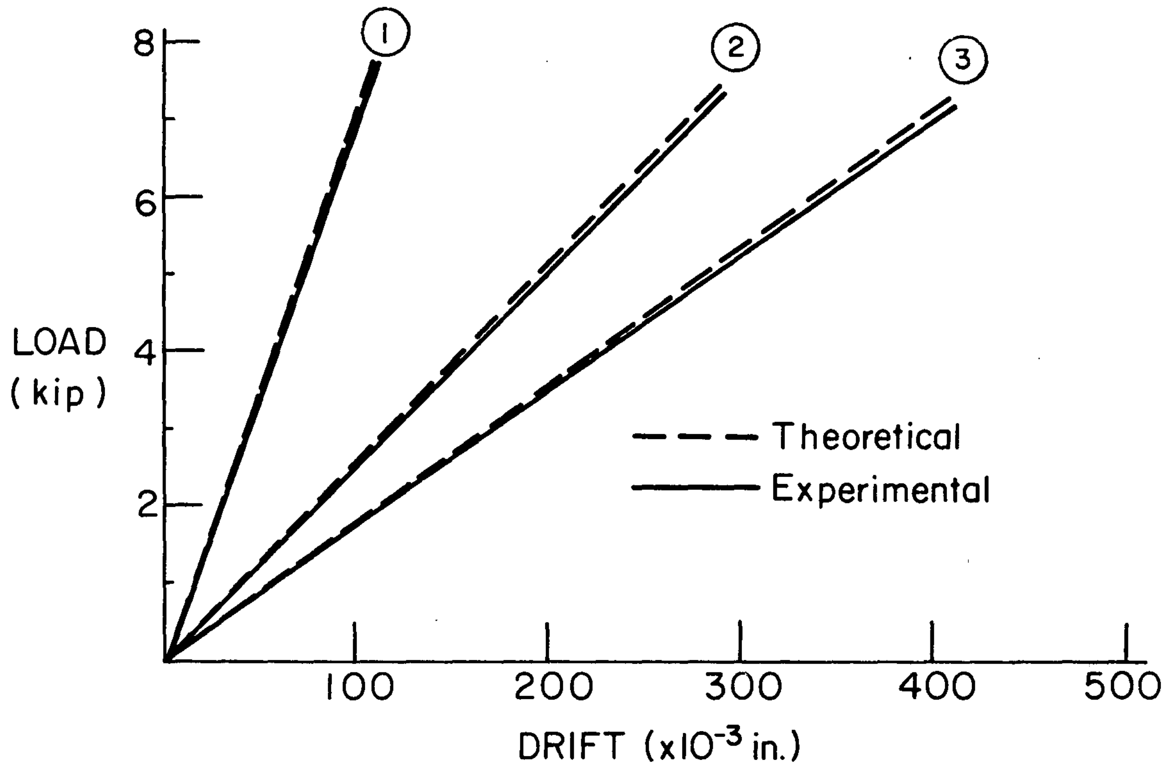


FIG. 32 DEFLECTIONS - TEST 3a

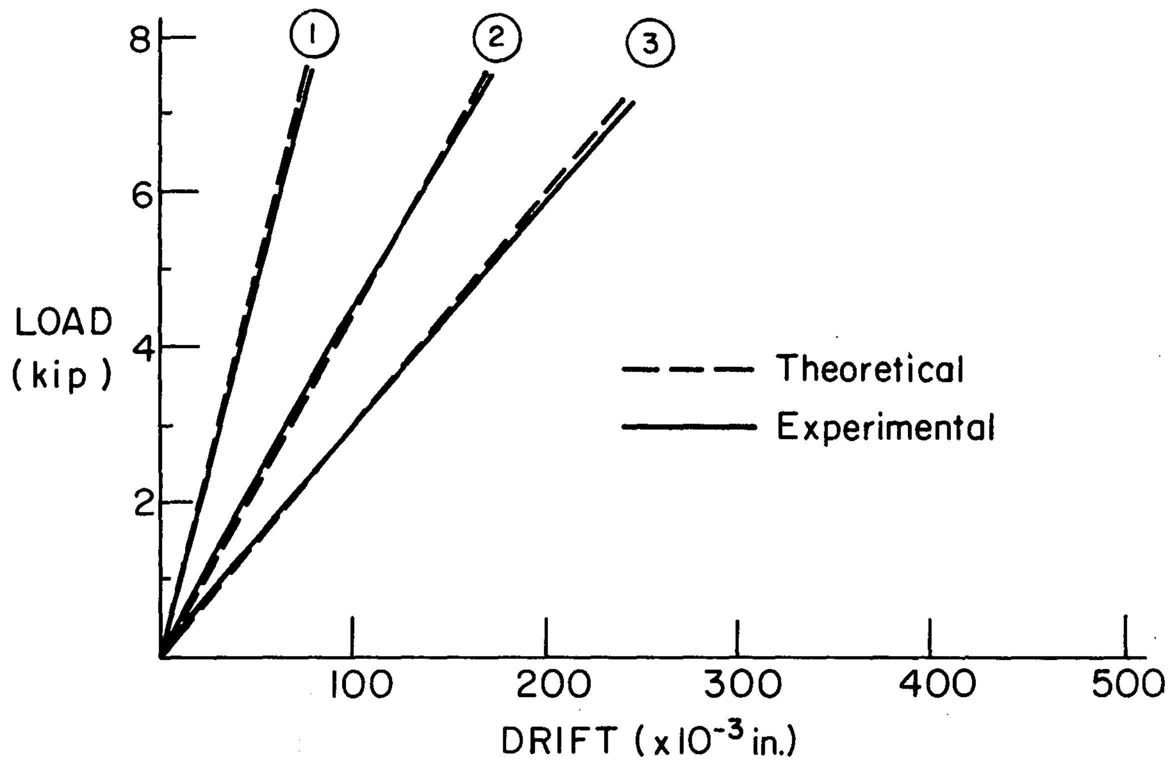


FIG. 33 DEFLECTIONS - TEST 3b

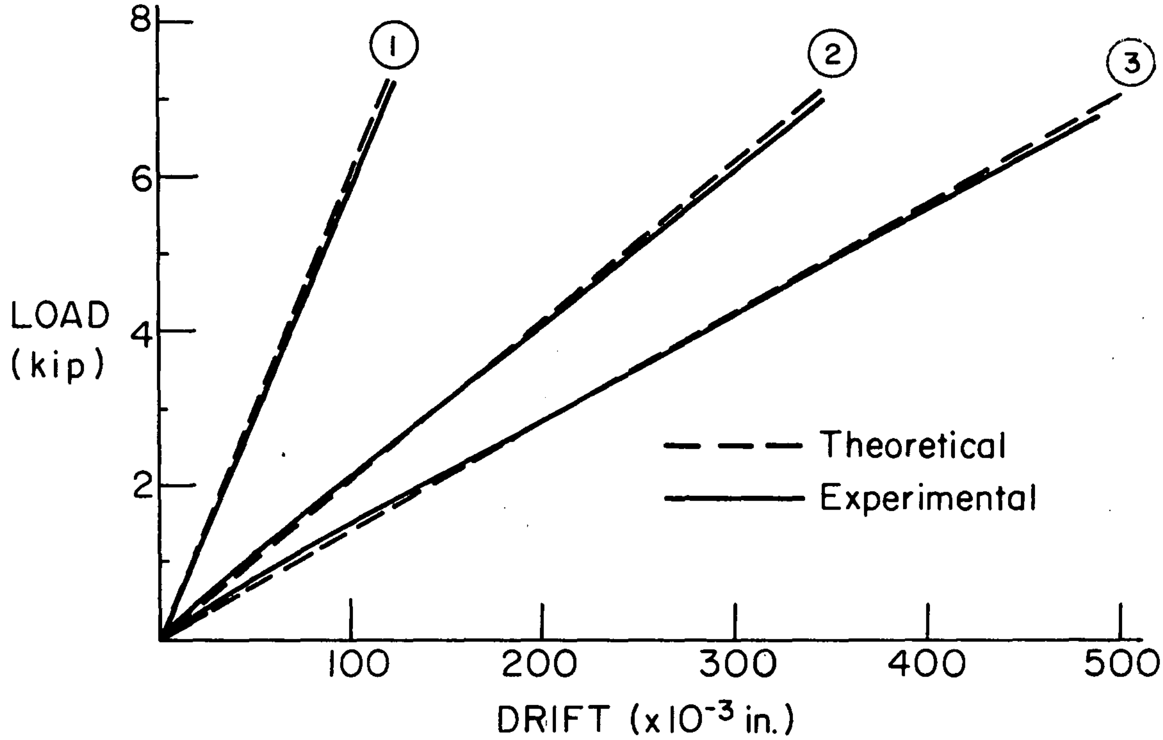


FIG. 34 DEFLECTIONS - TEST 3c

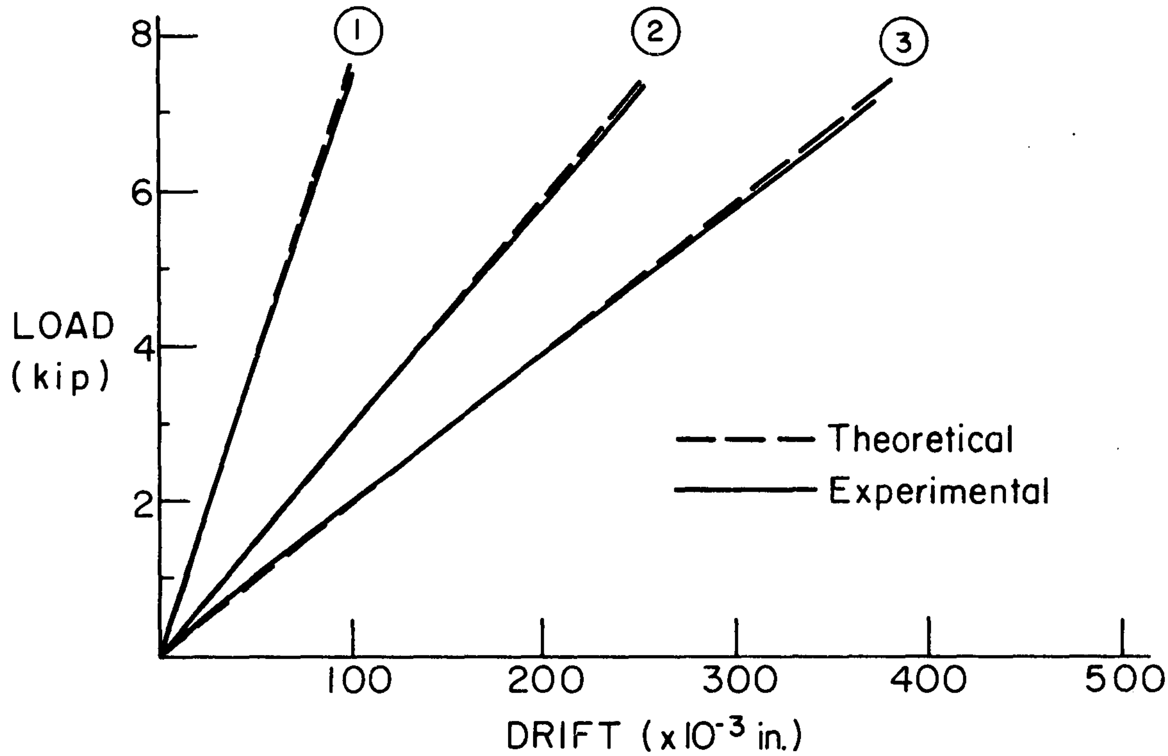


FIG. 35 DEFLECTIONS - TEST 3d

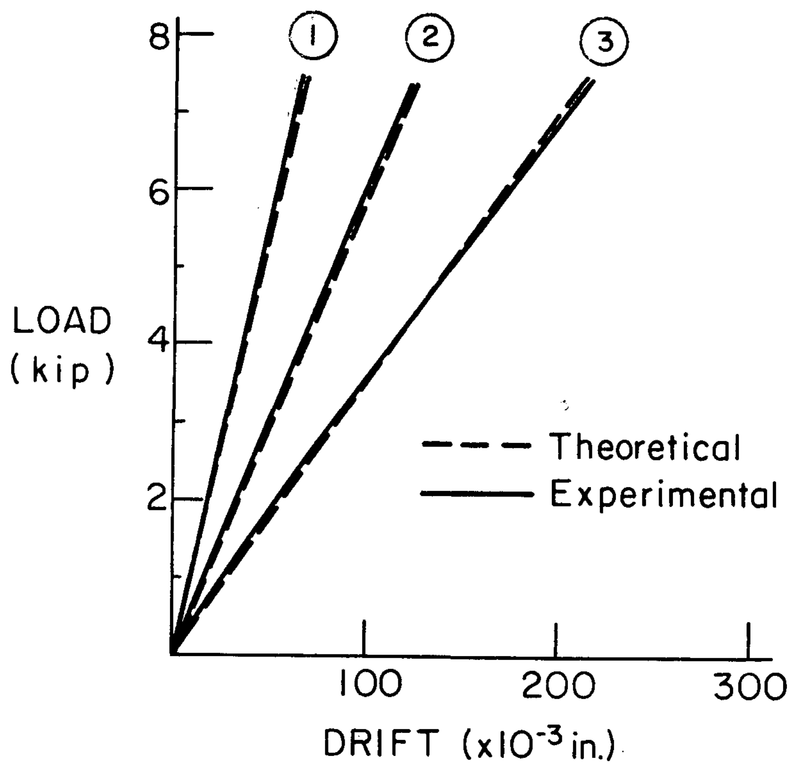


FIG. 36 DEFLECTIONS - TEST 3e

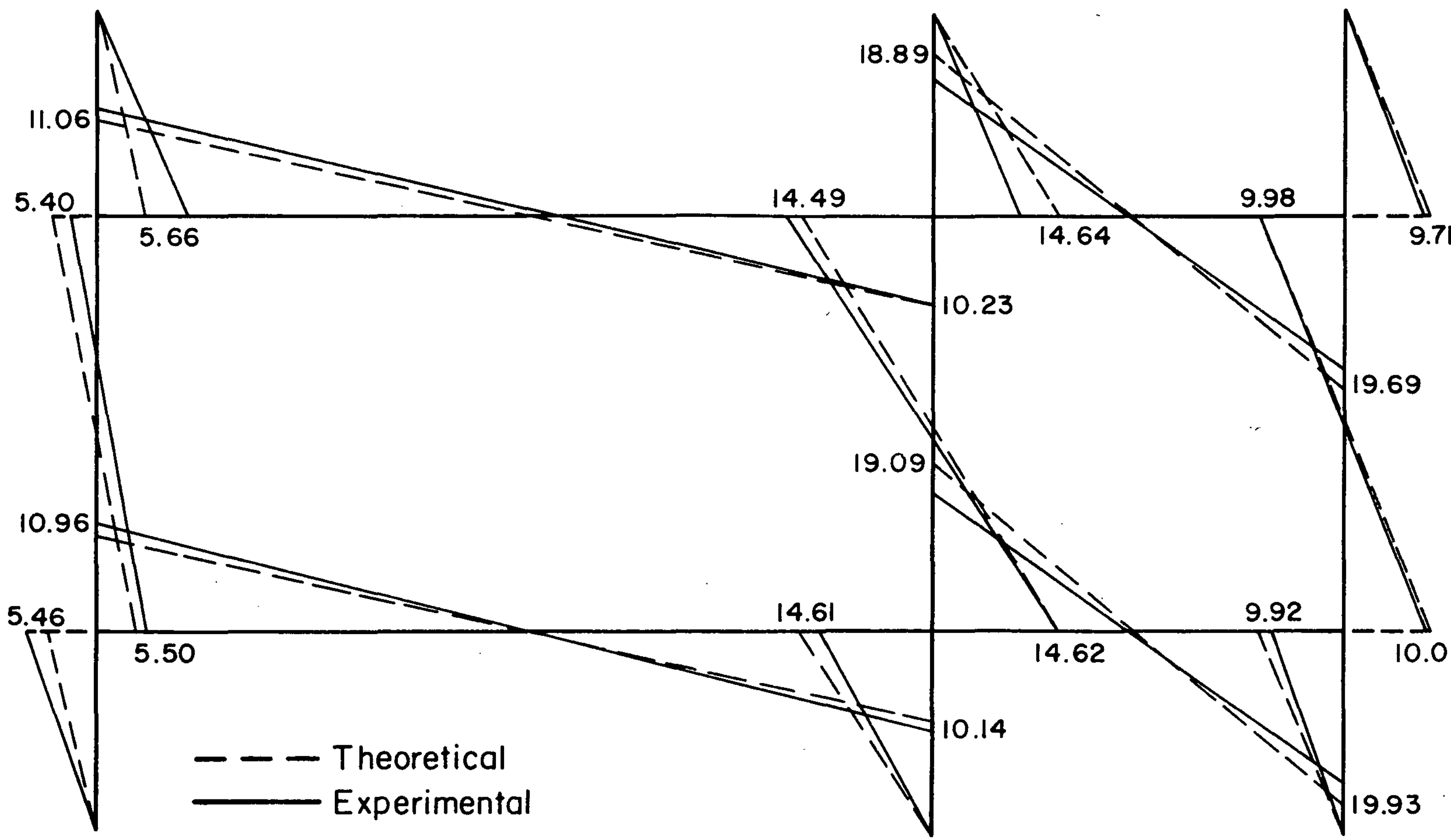


FIG. 37 BENDING MOMENTS - TEST 1

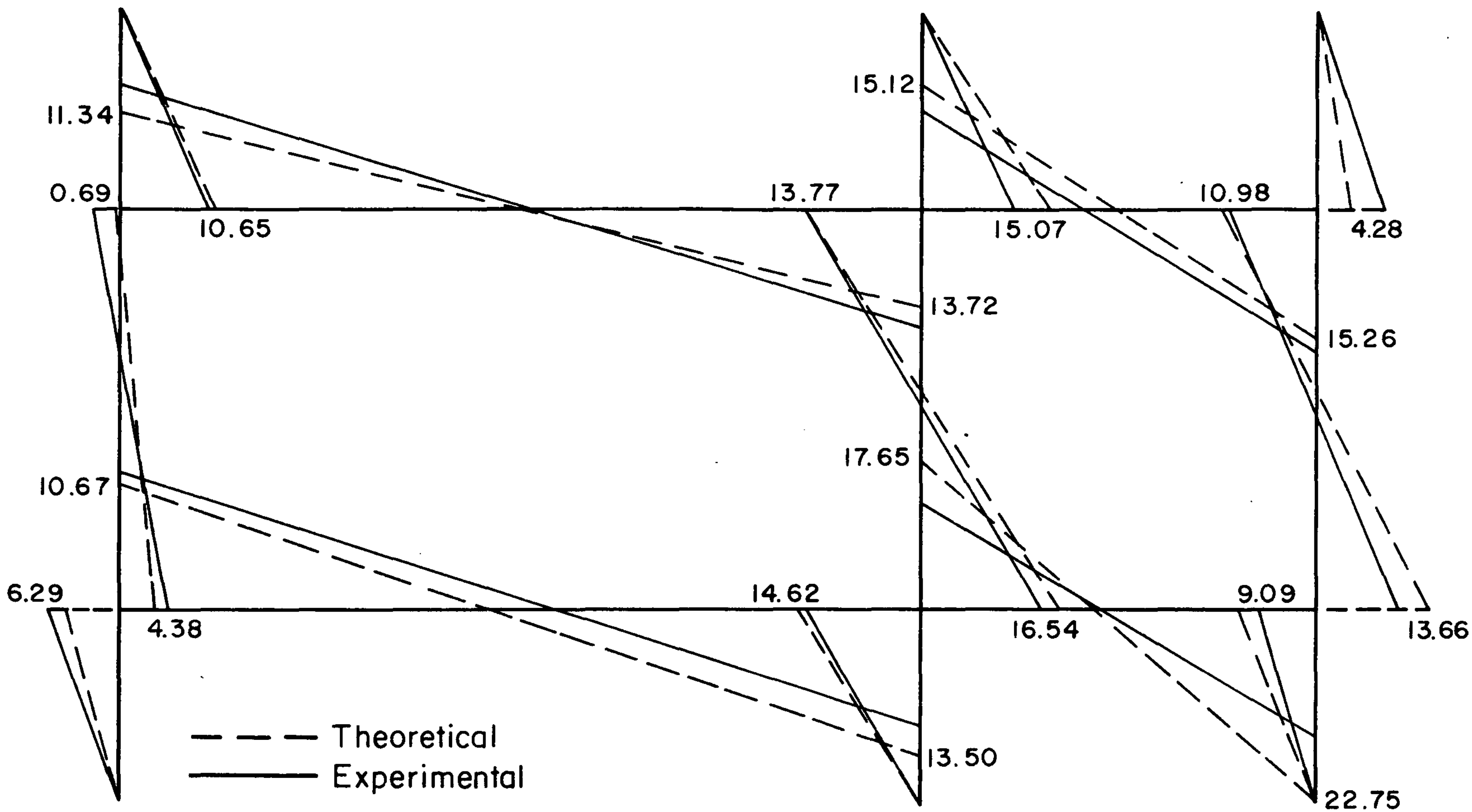


FIG. 36 BENDING MOMENTS - TEST 2c

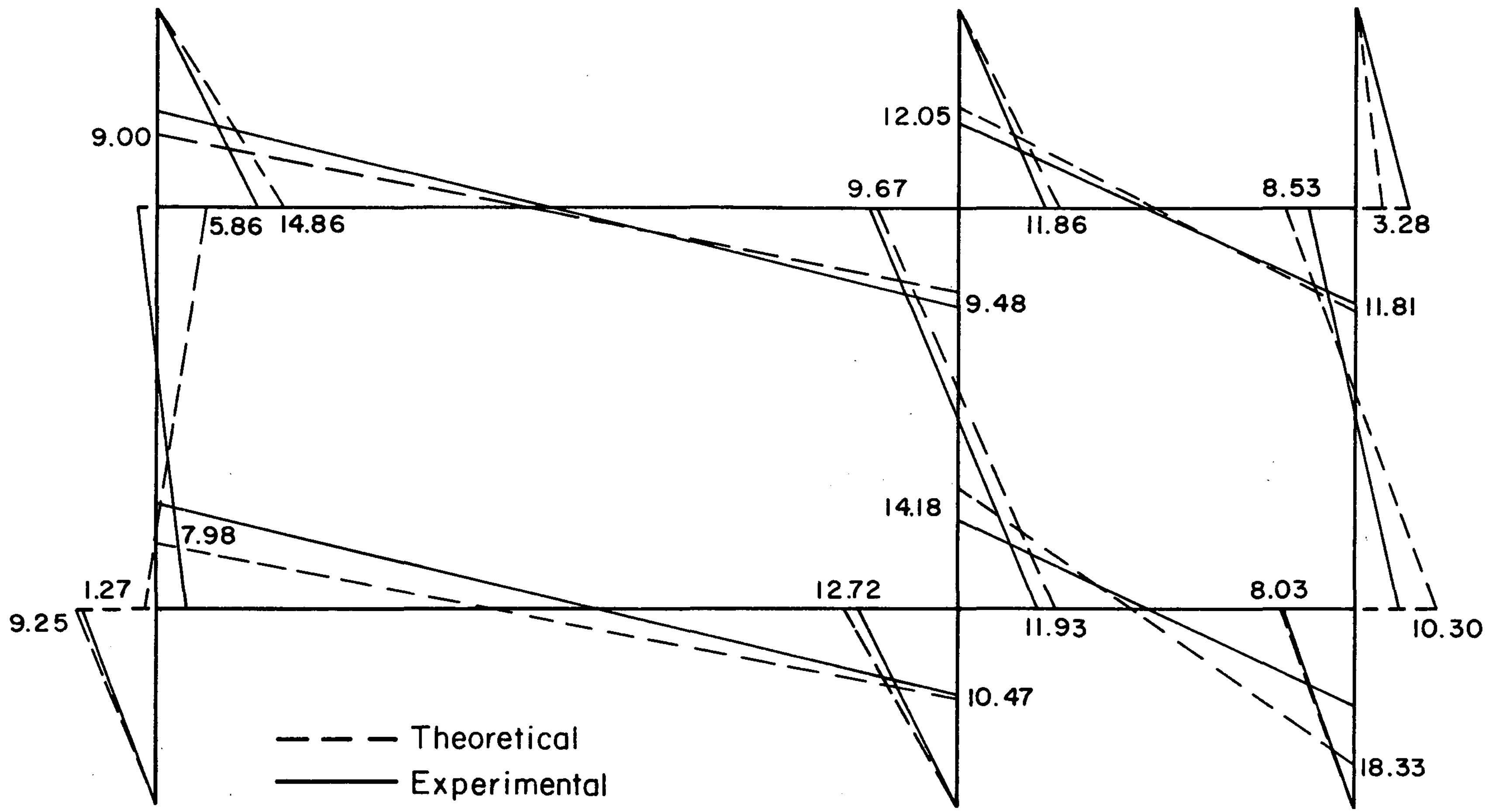


FIG. 39 BENDING MOMENTS - TEST 3a

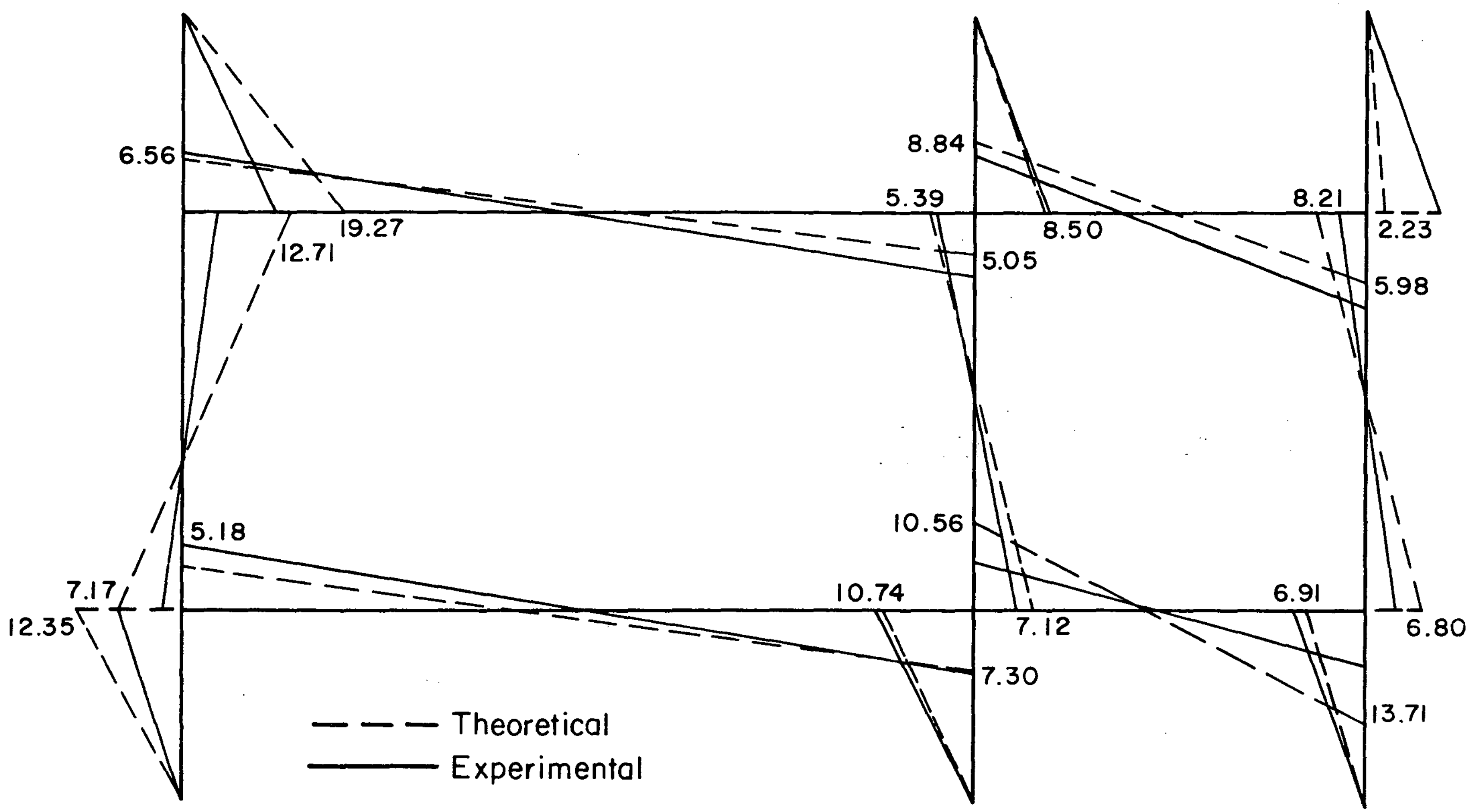


FIG. 40 BENDING MOMENTS - TEST 3b

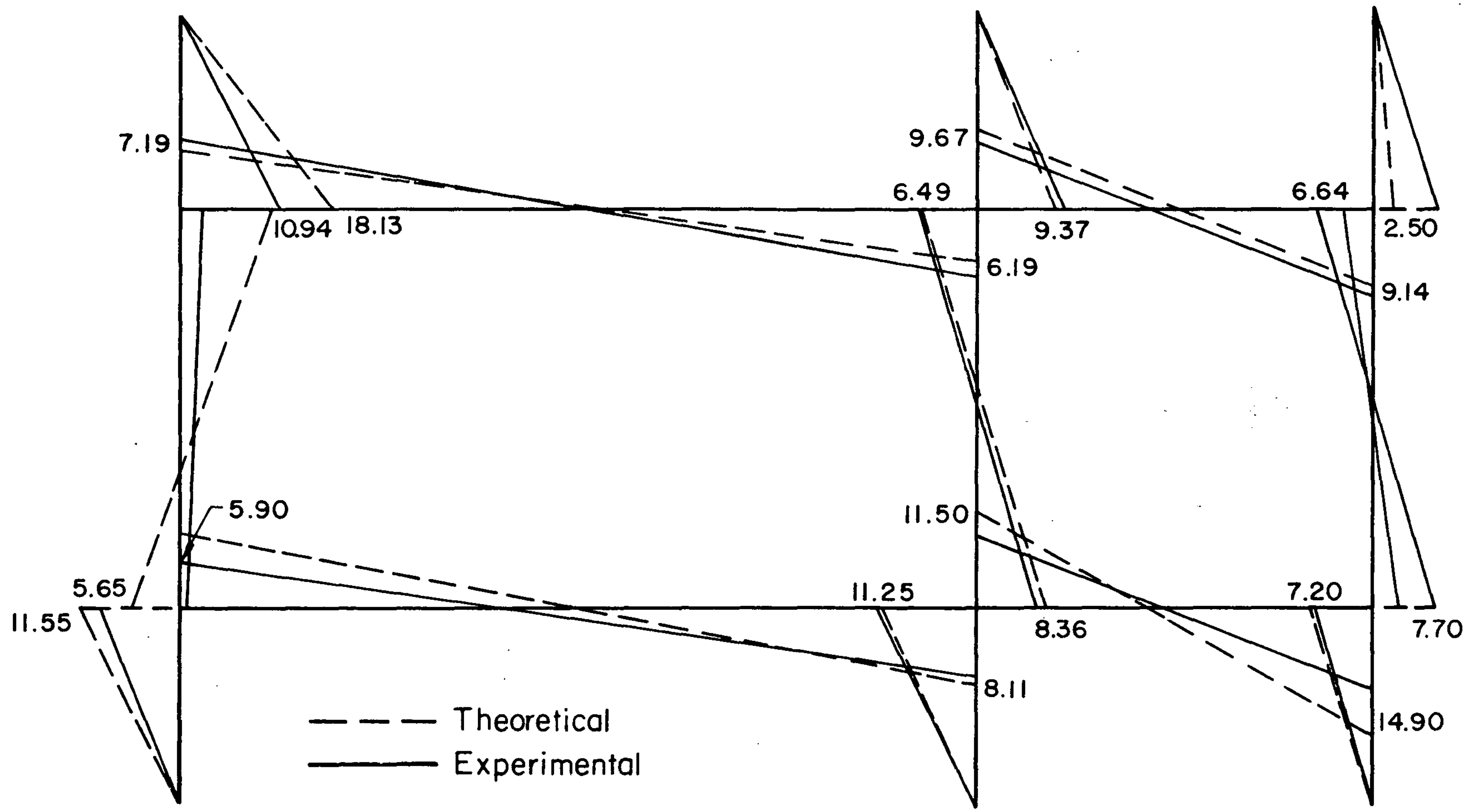


FIG. 41 BENDING MOMENTS - TEST 3c

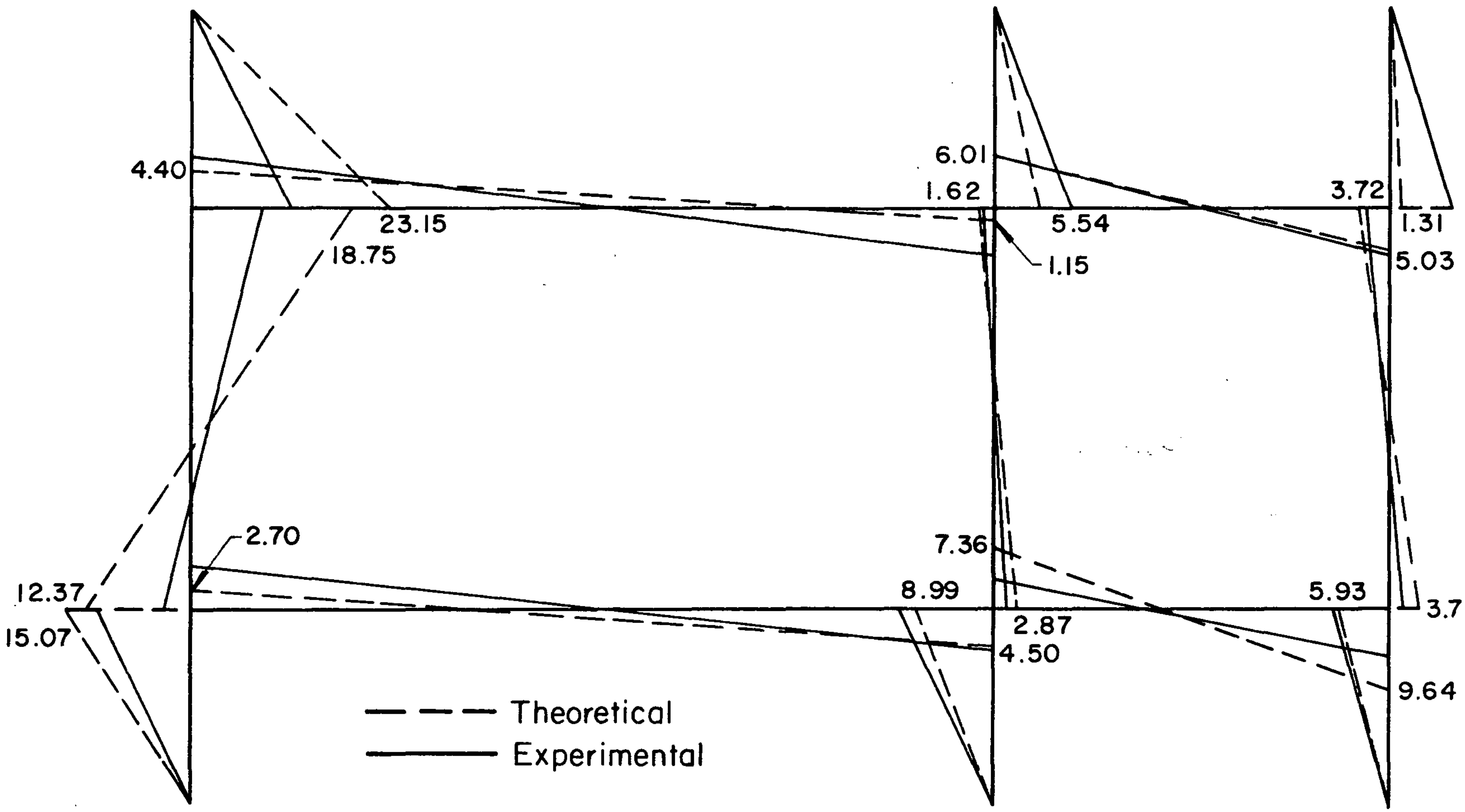


FIG. 42 BENDING MOMENTS - TEST 3d

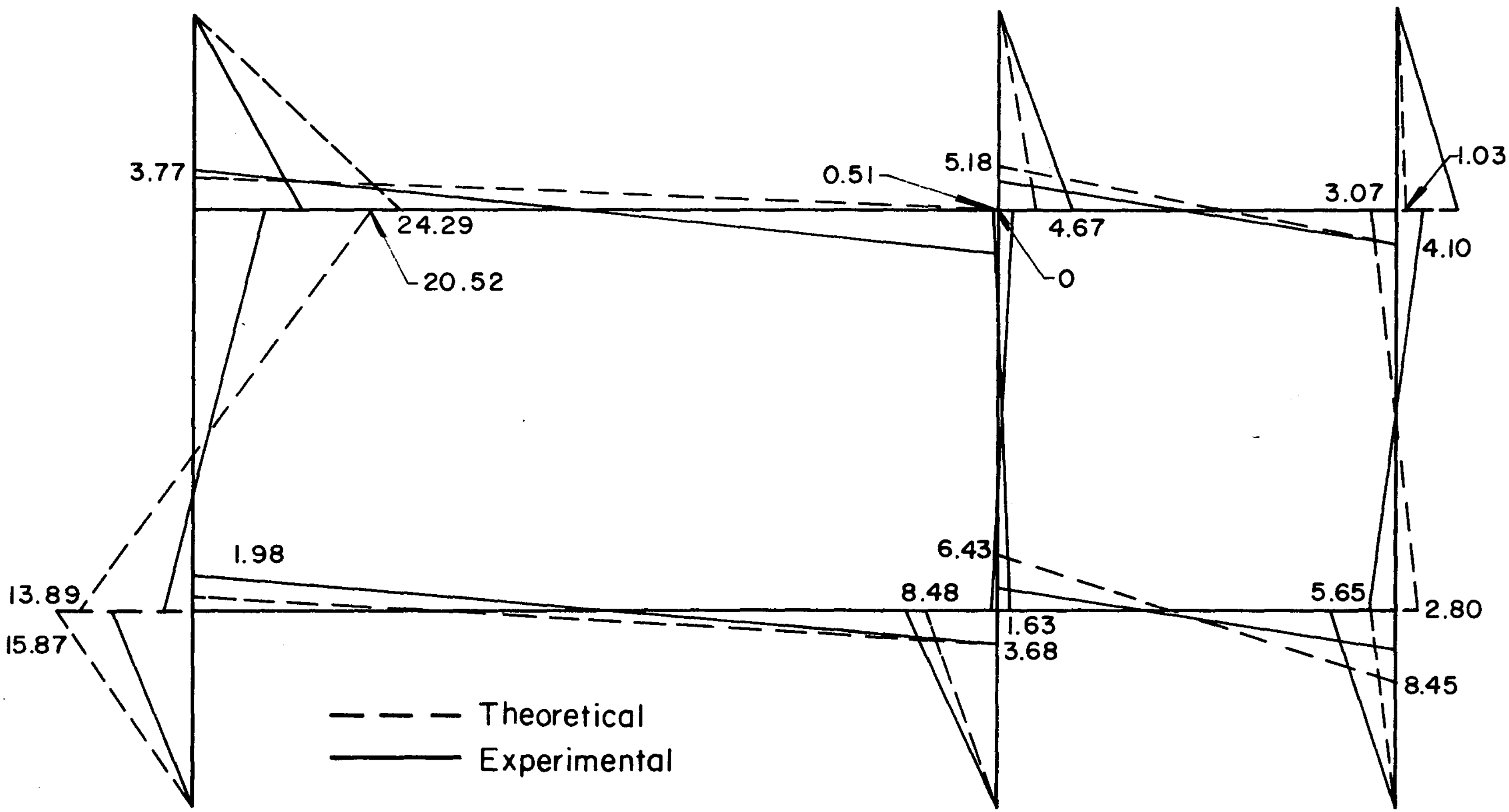


FIG. 43 BENDING MOMENTS - TEST 3e

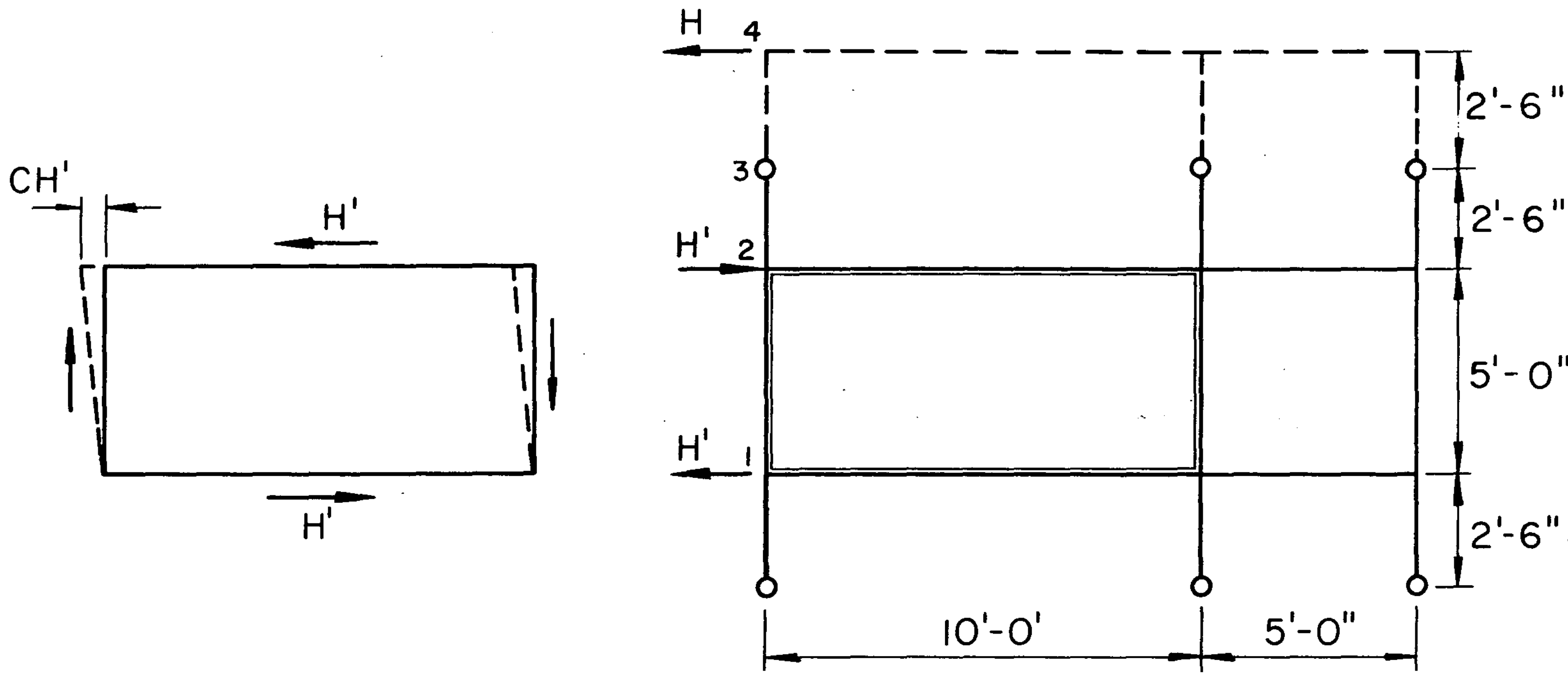


FIG. 44 FORCES ON CLAD TEST FRAMES

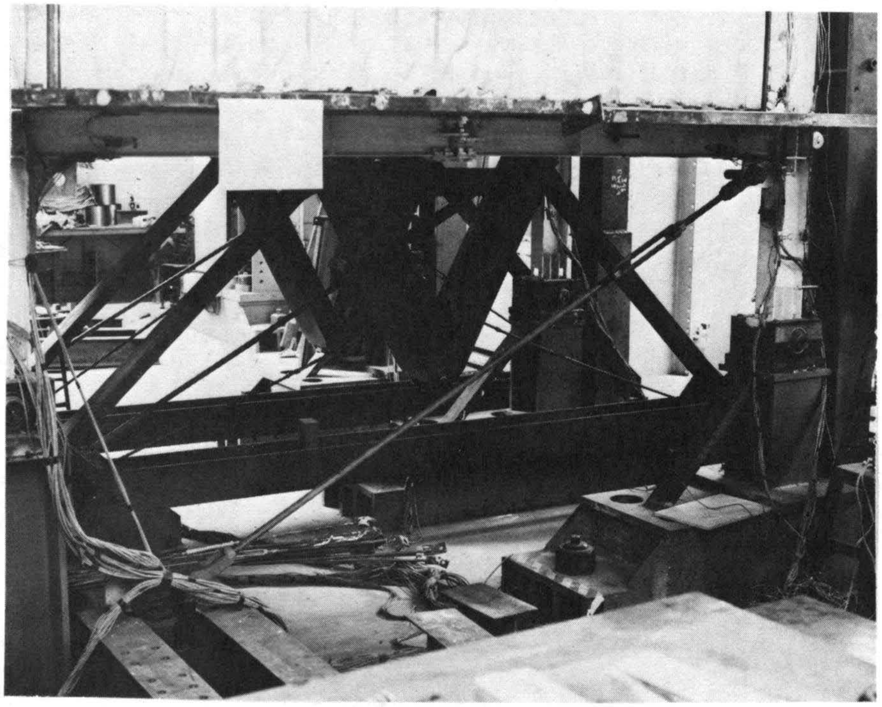


FIG. 45 DIAGONAL BRACING OF BOTTOM STORY

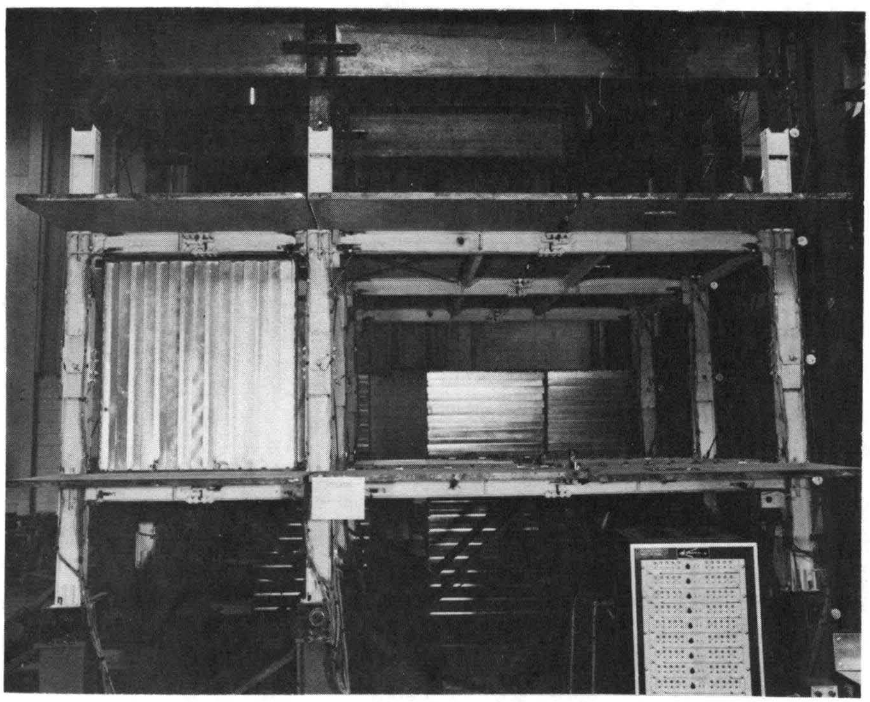


FIG. 46 TEST BUILDING WITH PARTITIONS IN 5-FT. BAY

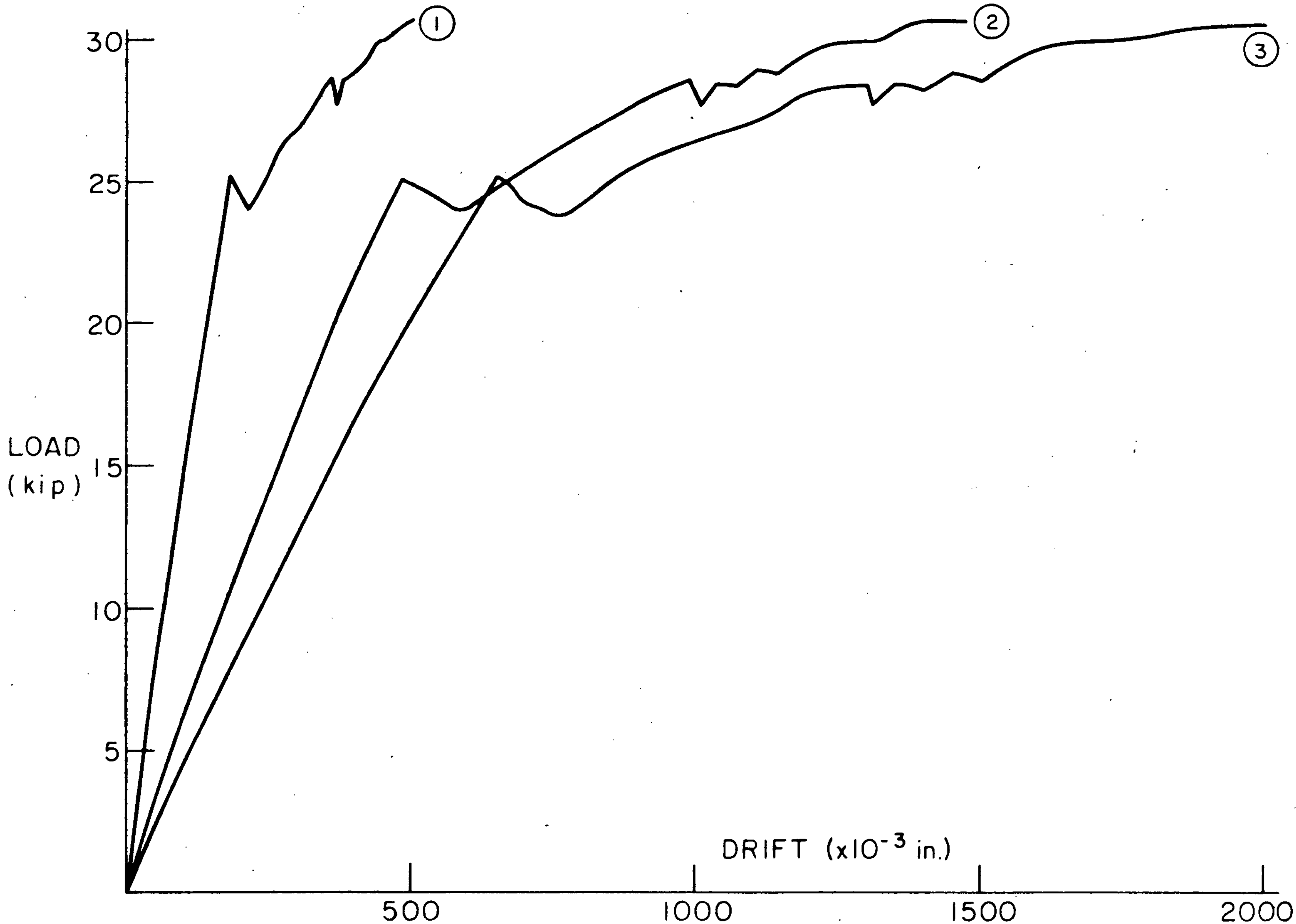


FIG. 47 LOAD-DRIFT OF CLAD FRAME (PARTITION IN 10-ft. BAY)

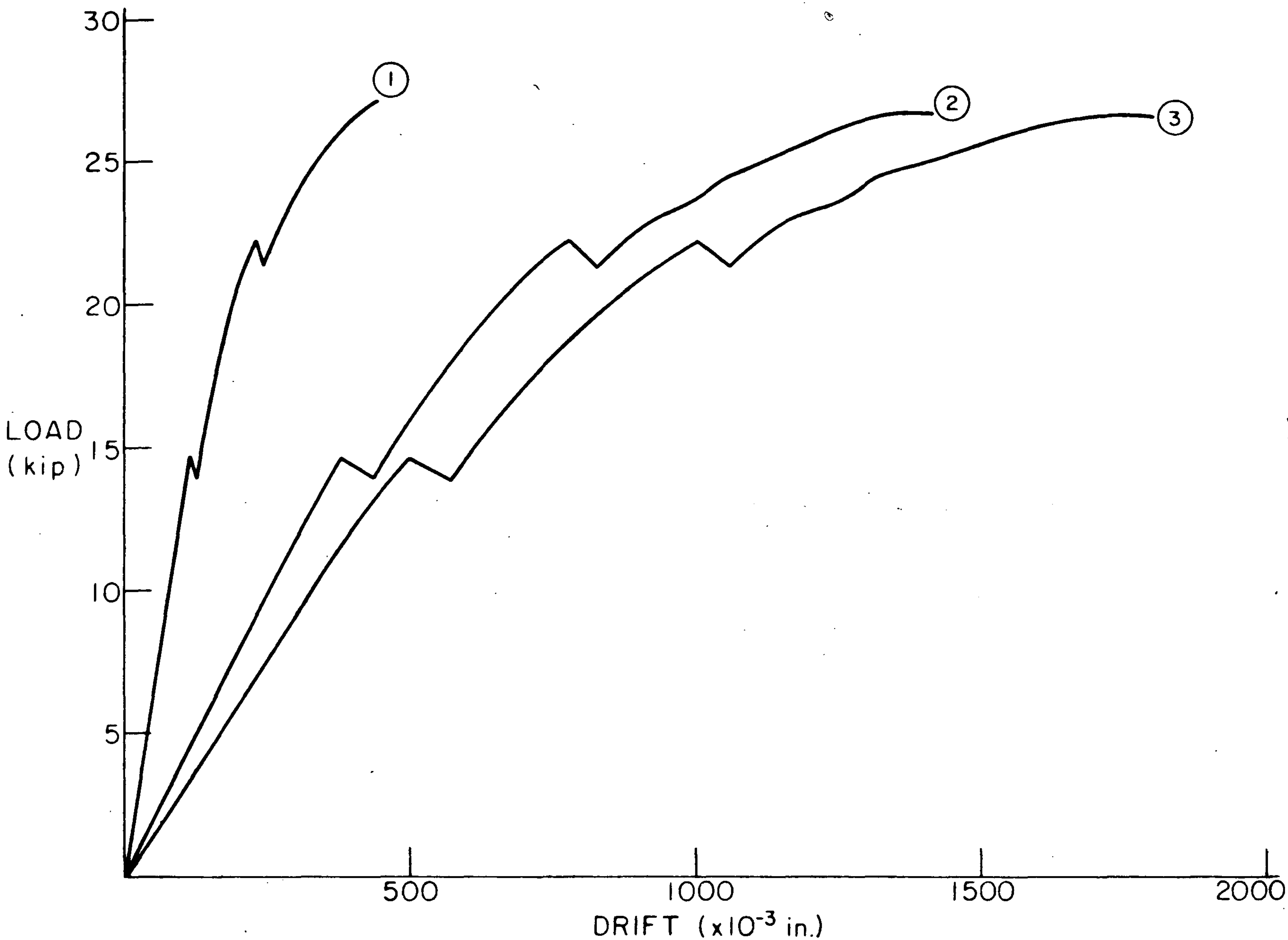


FIG. 48 LOAD-DRIFT OF CLAD FRAME (PARTITION IN 5-ft. BAY)

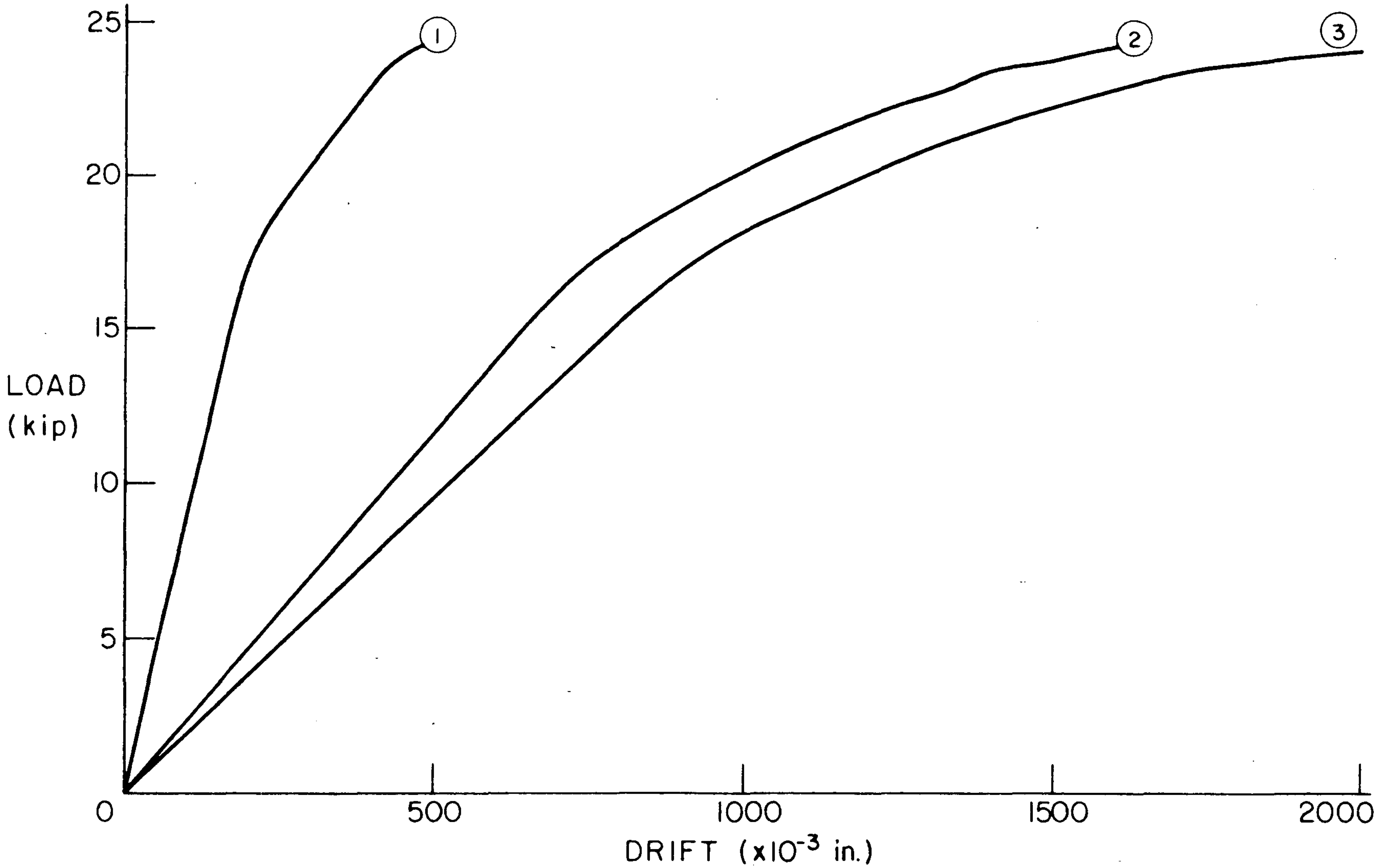


FIG. 49 LOAD-DRIFT OF BARE FRAME

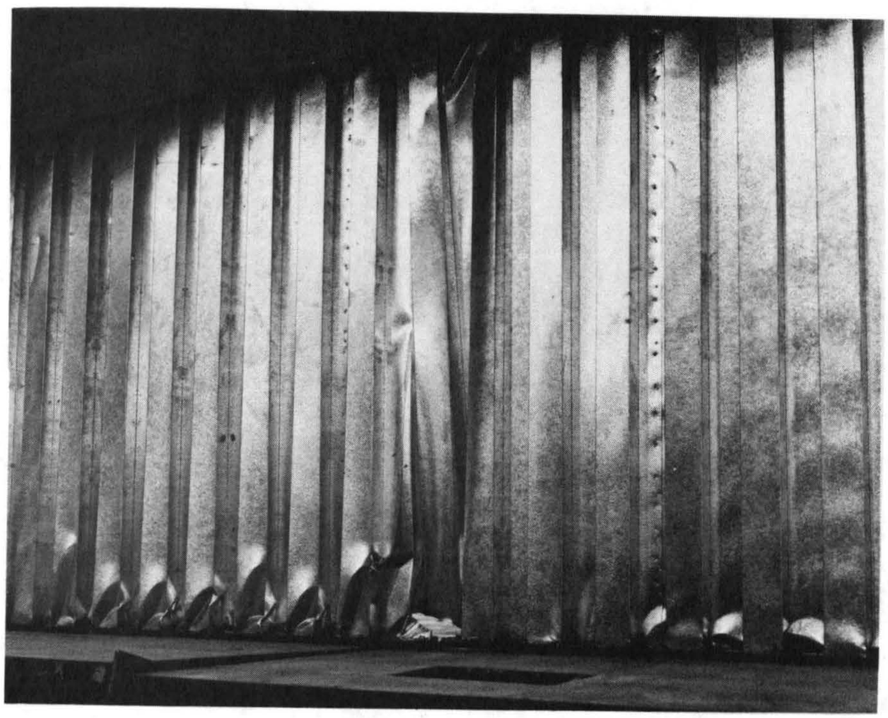


FIG. 50 FAILURE OF 10-ft. BAY PARTITION

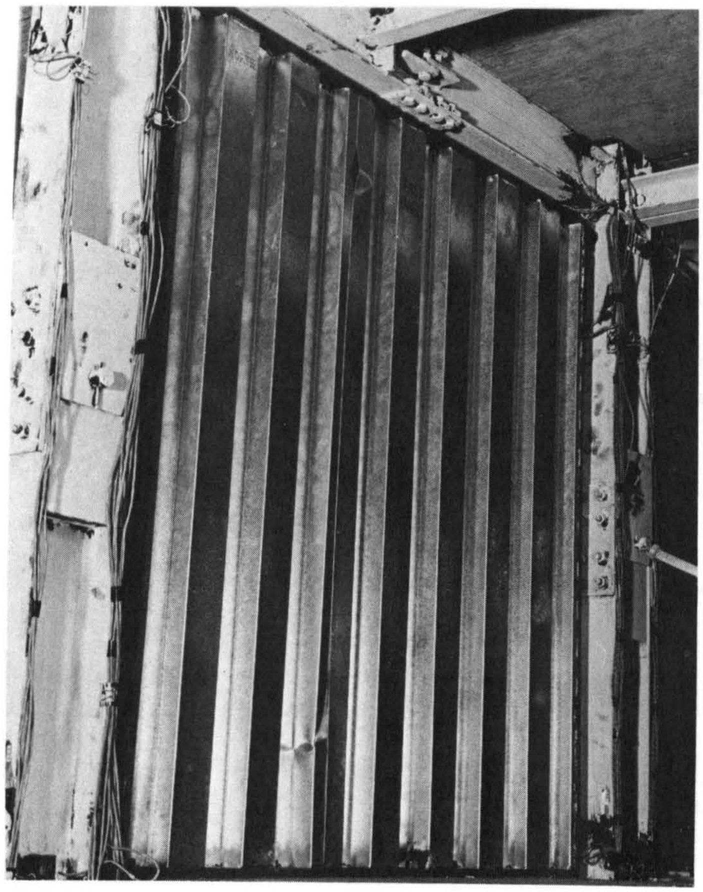


FIG. 51 DEFORMATION @ SEAM FASTENERS

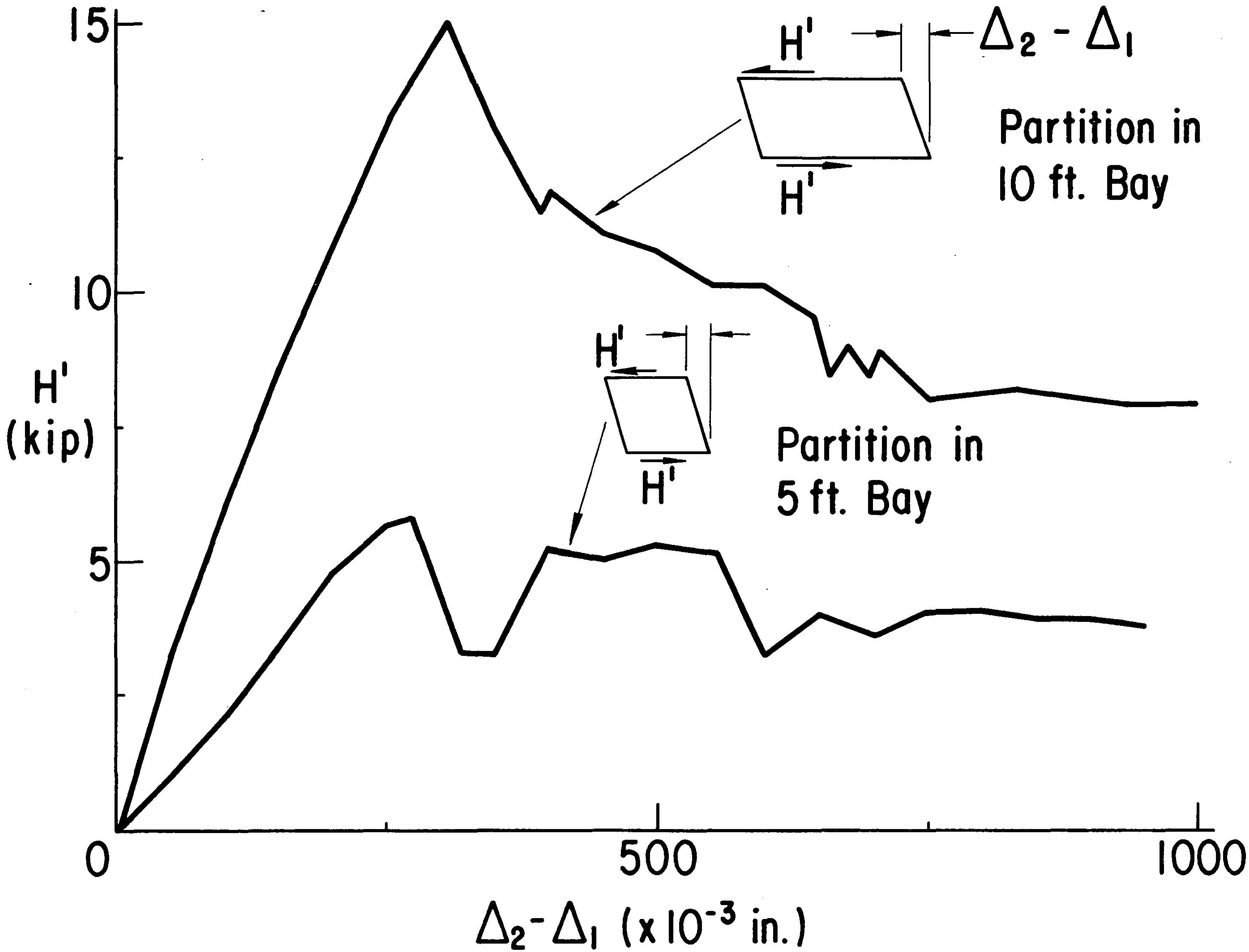


FIG. 52 SHEAR-DRIFT BEHAVIOR OF STRUCTURAL PARTITIONS

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