米日	U.S. - JAPAN COORDINATED PROGRAM FOR MASONRY BUILDING RESEARCH

REPORT NO. 5.2-1

PB93-214641

ASSEMBLY OF EXISTING DIAPHRAGM DATA

by

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JULY 1990

supported by:

NATIONAL SCIENCE FOUNDATION
GRANT NO. CES-8517028
FORMERLY GRANT NO. ECE-8517028

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This report presents the results of a research project which was part of the U.S. Coordinated Program for Masonry Building Research. The program constitutes the United States part of the United States - Japan Coordinated Masonry Research Program conducted under the auspices of the Panel on Wind and Seismic Effects of the U.S.-Japan Natural Resources Development Program (UJNR).

This material is based on work supported by the National Science Foundation under the direction of Program Director, Dr. S.C. Liu.

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Research Contract No. ECE - 8517028

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ABSTRACT

Masonry buildings are subjected to earthquake and other lateral forces. These loads must be distributed through the different structural components. In masonry structures the floor diaphragm transmits these loads from one wall to another. Adequate diaphragm action is necessary for the floor system to absorb or transmit this dissipated energy. Proper connections between the diaphragm and the masonry walls, as well as between individual diaphragm elements, are essential to the development of this diaphragm action. Other parameters affecting the diaphragm behavior are its thickness, orientation, and (in the case of precast concrete elements) the addition of topping, and the number of boundary sides connected. In short, the structural integrity of the masonry buildings is linked directly to the resistivity of the diaphragm to lateral in-plane loads.

Category 5.0 of the U.S. - Japan coordinated program for masonry research is a detailed investigation of horizontal diaphragms. The objective of this research task (5.2) is to assemble extensive experimental data on various types of floor diaphragms and to reduce the data to a form required for static and dynamic analysis models. An extensive bibliographic search was done on cold-formed steel decks, composite steel decks, timber diaphragms, reinforced concrete diaphragms, and precast prestressed concrete diaphragms. Equations required to predict the initial stiffness and peak strength (parameters related to the EKEH model) for steel deck reinforced concrete, cold-formed steel, precast prestressed, and plywood diaphragms are included. Experimental data related to the Lumped Parameter Model (EKEH Model) is also presented.

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1. INTRODUCTION

1.1 General Remarks

Buildings subjected to earthquake and wind forces must have floors and roofs capable of transferring in-plane shear forces from one wall to another. These floors are termed as diaphragms and are typically made of hollow core planks, reinforced concrete, composite steel deck reinforced slabs, or timber (See Appendix B for a particular description on each type of diaphragm). Diaphragms are relatively thin, usually rectangular, structural elements capable of resisting shear parallel to their edges. They must be used as walls in a vertical position (shear walls), or as roofs or floors in a horizontal position. Diaphragms may be considered to be a horizontal beam with the roof or the floor system acting as the web of the beam, the joists or beams function as web stiffeners, and the peripheral beams or integral reinforcement function as flanges.

The function of the diaphragm is to brace a structure against lateral forces, such as wind or earthquake loads, and to transmit these forces to the other resisting elements of the structure. Figure 1 illustrates the distribution of lateral forces on a typical structure. The distribution of lateral forces to the shear wall or space frame systems depends on the properties of the diaphragm and the resisting system.

In order to improve the performance of the floor and roof systems, it is desirable for the in-plane stiffness of the diaphragm to exceed that of their respective vertical subsystems [90]. Diaphragms of this type have been categorized as rigid. In this instance, the diaphragms act as a flat plate that transmits lateral loads to the bracing elements in proportion to their relative rigidities. Conversely, with flexible diaphragms, loads are distributed to vertical subsystems as a continuous beam using tributary areas. Regardless, both rigid and flexible systems should be able to retain a sufficient amount of in-plane stiffness or strength in order to prevent collapse, well beyond the elastic range [376].

1.2 Objective of the Overall Research Program

The research undertaken for this project is part of the U.S.-Japan Coordinated Program for Masonry Building Research. Each category of this program is conducted under the supervision of the Technical Coordinating Committee for Masonry Research (TCCMAR). The TCCMAR committee was organized to function under the auspices of the Panel of Wind and Seismic Effects of the U.S.-Japan Cooperative Program in Natural Resources (UJNR). Study of floor diaphragms, which is the objective of this project, is the fifth research task. Additional information of the organization of the Masonry Building Research Program is available in Reference [348].

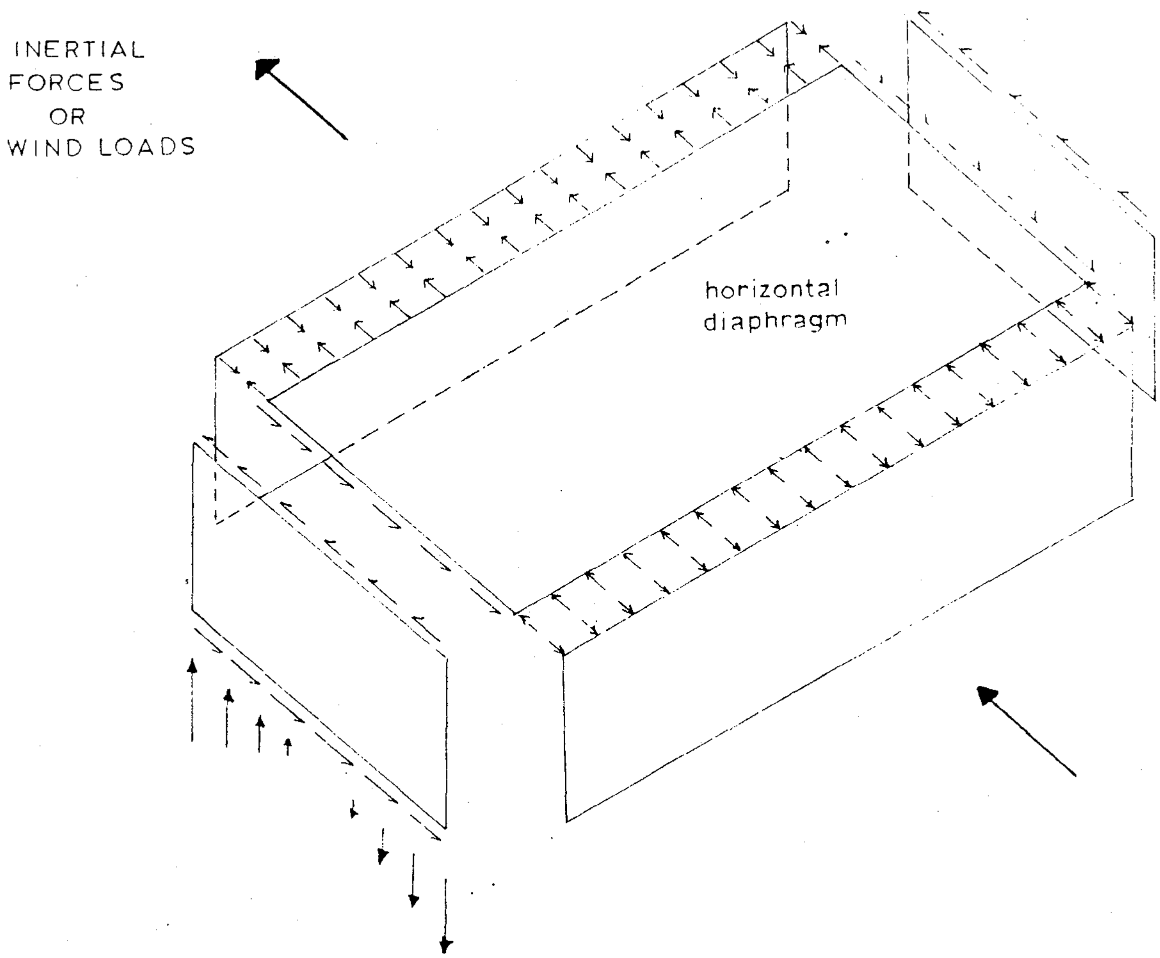

Figure 1. Distribution of lateral forces on a structure

The study of diaphragm characteristics was divided into two separate tasks. Task 5.1 involved both experimental and analytical investigation of precast horizontal diaphragms subjected to in-plane loading. Task 5.2 focused on the collection of existing literature and data generated from the discussion and testing of horizontal diaphragms. The subject matter of this report focuses on Task 5.2 only.

The objectives of the overall Task 5.0 research project were to determine the basic failure modes, ascertain behavioral characteristics, and investigate analytical properties for the analysis of full-scale precast, prestressed hollow-core plank diaphragms subjected to in-plane shear. Basic analytical features of interest include determination of the force-deflection relations, first major event load, limit state strength, ductility, stiffness, failure modes, and hysteretic behavior.

In order to fulfill these objectives, the effects of various parameters were investigated (Task 5.1). These parameters include:

- boundary condition (number of sides connected to the loading frame)
- orientation (placement of the planks with respect to the direction of the applied lateral load)
- slab thickness (plank depth of six, eight and twelve inches)
- aspect ratio (geometric configuration of the diaphragm)
- topping (addition of a two-inch cast-in-place concrete slab)
- seam connectors (variation in the number of seam connectors to verify the implications of attaining an alternate failure mode for the untopped tests)

The second phase of the project (Task 5.2) entailed gathering and reviewing existing literature and data on horizontal diaphragms. The information collected from both Task phases 5.1 and 5.2 of the project is to be combined and used in other task areas of the Masonry Building Research Program which culminates with the eventual construction and testing of a full-scale masonry building.

1.3 Scope of the Project

In order to assemble and reduce existing experimental data on various types of floor diaphragms to a form required for static and dynamic analysis models, the following diaphragm items were searched:

a) Limit state strength
b) Initial stiffness
c) Failure mode type
d) Test identification
e) Type of testing

This document includes a summary of the basic data obtained from each of the items listed above. The purpose of this tabulation (Task 5.2) is to provide information to allow the formulation of a model for analysis of masonry buildings containing diaphragms. Section two contains review of previous work done on cold-formed steel, composite steel deck, wood, precast concrete, and reinforced concrete diaphragms, and also includes a review of hysteretic models. Section three provides a detailed description of the models used in the diaphragm research including the test rationale, brief description of the specimens, test setup, instrumentation, and description of the test data acquired which is reported in Appendix A. Section four contains the equations required to predict the initial stiffness and maximum load of composite steel deck, cold-formed steel, precast concrete, and plywood diaphragms. A summary of this project is contained in Section five. Section seven presents an exhaustive list of references.

2. REVIEW OF PREVIOUS WORK

2.1 General

A well-designed diaphragm is essential for the structural integrity of a building subjected to earthquake induced motions or wind forces. The distribution of in-plane forces from the diaphragm to the various elements of the vertical load resisting system depends on the properties of the diaphragm and the vertical load resisting system. Thus, knowledge of the behavior characteristics of a diaphragm is necessary to perform a lateral load (seismic) analysis of a multi-story building [404].

Diaphragms may be categorized according to their composition into the following common types: cold-formed steel, composite steel deck, timber, reinforced concrete, and precast concrete. The seismic performance of each of these systems is different and depends on the characteristics of the diaphragm and the event.

This chapter will review past research that relates to the behavioral and design of each one of the systems listed above. The following sections address studies of cold-formed steel, composite steel deck, timber, reinforced concrete, and precast concrete.

2.2 Cold-Formed Steel Diaphragms

The following review of steel-deck diaphragm research was partially taken from Section 2.3, pages 10-12, of Reference [378]. It was written by Max L. Porter, P.E. and W. Samuel Easterling.

Early testing of steel deck diaphragms established behavioral characteristics which are keys to understanding and attempting to predict strength and stiffness. Nilson [342] provided the first generally published article on steel deck diaphragm research which was performed at Cornell University. In this research Nilson established techniques for testing steel deck diaphragms which were published by the American Iron and Steel Institute (AISI) [14] and have since been accepted by the American Society for Testing and Materials (ASTM) [33]. He also developed and refined several welding techniques particularly suited to steel deck sections.

Diaphragm research at Cornell University and West Virginia University was done by Luttrell [301,302]. In addition to deriving a semi-empirical expression for prediction the shear stiffness of a steel-deck diaphragm, he concluded that the panel length influenced shear stiffness but not strength. Luttrell noted that warping of the profile occurs near the ends of the panel and extends for a finite distance towards the panel center. This warping can be decreased by adding additional fasteners or by stiffening the ends as is done with closure angles [303]. He also stated that the length of the warped region is independent of the panel length.

A series of about 160 full-scale shear diaphragm tests were done by Luttrell and Ellifritt at West Virginia University [308]. Numerous test parameters were varied. these parameters included welded fasteners, thickness of sheets, panel width, panel length, purlin spacing, and fastener spacings. They derived empirical equations to predict the stiffness and strength of the shear diaphragms.

Luttrell and Huang [309] worked on a summary of deck diaphragm studies done at Major Units Laboratory of West Virginia University (WVU) over the past fifteen years. The work done at WVU covered tests on a wide range of deck panel types, metal building wall systems, panels with insulating concrete fill, and plywood assemblies, all used as diaphragms, and under static, cyclic, and dynamic loading. Connection performance was evaluated through an extensive test program covering arc-spot welds, different types of screws, and power driven pins. This study was the support document for the 1981 and 1987 Steel Deck Institute Diaphragm Design Manual [306,307].

Numerical modeling of steel deck diaphragms using finite analysis was performed at Cornell University [343,38]. A linear elastic analysis was used by Nilson and Ammar [343] in which steel deck panels were modeled as one-dimensional linear beam elements and connections were modeled with two one-dimensional elastic link elements. The mechanical properties for the plane stress elements and the linkage elements were determined by either experimental or analytical means.

Analytical models and comparisons to past experimental work were made using tests by Nilson [342] and Luttrell [301] according to Atrek and Nilson [38]. Due to nonlinearities in connectors, which could not be adequately modeled with the software used by Ammar, the analytical results were compared with the experimental results up to 40 percent of the experimental ultimate load. According to Atrek and Nilson [38] stiffness comparisons from Ammar's work were satisfactory for cellular type diaphragms, but for noncellular diaphragms, the analysis showed a more flexible result than the experimental specimens. This result was attributed to poor evaluation of the effective shear modulus of the deck.

Atrek and Nilson [38] expanded the finite element analysis that was begun by Ammar to include the nonlinear behavior of the connections. In addition, they considered different approaches for determining equivalent orthotropic properties for the steel deck such that plane stress elements could be used in the analysis. These different approaches were developed by other researchers and are referenced extensively by Atrek and Nilson [38]. Work by Hussain and Libove [235] of Syracuse University was utilized by Atrek and Nilson for determining the shear modulus.

In their work Atrek and Nilson reiterated the fact that the majority of the shear deformation occurs near the panel ends in steel-deck diaphragms. As part of their conclusions, they stated that increased accuracy can be obtained when comparing the analytical results to the experimental results by assigning the elements near the ends of the diaphragm a reduced effective shear modulus compared to the centrally located elements. However, they also conclude that the increase in accuracy does not, in most cases, overcome the added expense that comes with such mesh refinement.

A series of 14 full-scale diaphragm tests were performed and reported in 1981 by ABK, a joint venture [2,3]. Of the 14 diaphragms, two specimens were cold-formed steel deck diaphragms. The specimens were 20-gage steel decking, unfilled, the first one was unchorded with button-punched seams 6 in. o.c. Each specimen was attached to a 20-ft by 60-ft test frame. Loading consisted of a series of reversed cyclic, quasi-static and dynamic inputs with each series generally increasing in magnitude. The document is a data report on the experimental testing.

Twenty-two diaphragm tests were performed at Iowa State University by Lukens [299]. Each diaphragm consisted of 29-gage steel cladding on a timber frame. Numerous test parameters were varied. These parameters included diaphragm length, purlin wood type, nail pattern, and loading type. Of the 22 diaphragms, four specimens were subjected to reverse loading for a defined number of cycles and then were subjected to monotonic load to the ultimate load. As part of his conclusions Lukens stated the following: first, the panel strength had a tendency to be independent of length when failure was caused by the purlin to chord connection splitting the chord. Second, panels with Douglas fir-larch purlins were about 20% stronger at failure load than panels with spruce-pine-fir purlins. Third, panels with nails through the ribs were about 40% less stiff than panels with screws in the valleys. Finally, reverse loading panels were about 12% weaker at failure load than nonreversed loading panels.

2.3 Composite Steel Deck

The following review of steel-deck-reinforced concrete diaphragms (SDRC) was partially taken from Section 2.2, pages 7-9, of Reference [378]. It was written by Max L. Porter, P.E. and W. Samuel Easterling.

Prior to the studies by Porter, et. al. [378,390] at Iowa State University (ISU),the quantity of general research done on SDRC diaphragm systems had been minimal. Most of the work that had been done was proprietary testing that was typically sponsored by various steel deck manufacturers. As a result of the proprietary nature of the work, information regarding the testing has not been widely published, but has been incorporated into various design methods.

In reviewing the previous SDRC diaphragm research, both from the limited publishing and proprietary data reports, Porter and Easterling [378] concluded that a substantial amount of the SDRC diaphragm testing was performed by S. B. Barnes and Associates of Los Angeles, California [44-46]. Since the late 1940's numerous proprietary tests have been performed for several different steel-deck manufacturers. The results of some of these early tests were used to formulate the design approach in the Seismic Design for Buildings [127,128].

Discussions of the test results [44-46] indicate the predominant modes of failure were deck tearing around welds at the edge framing member, shearing of welds, shear failure of the concrete above the flutes of deck, localized cracking and concrete separation from the deck. In most citations of the separation of the deck and concrete, a simultaneous failure of the welds was noted. Diaphragms with both structural concrete fill and vermiculite fill were tested with a minimum of 1-1/2 inches of concrete above the top flange of the deck. Steel deck sections were typically welded to the framing members.

Based on some of the early tests by Barnes, a design method was developed using a guided cantilever concept and basic statics [29]. Resulting expressions were empirical and calibrated to fit the limited number of tests that were available at the time. The equations were typically adjusted for each different deck manufacturer's tests [127,128]. Methods for both strength and stiffness were formulated based on all available tests prior to the 1973 publishing of Seismic Design for Buildings [127,128].

A series of nine diaphragm tests with lightweight insulating fill were performed at WVU and reported by Luttrell [304]. Corrugated galvanized steel deck was used with nominal depths between 9/16 in. and 1-3/8 in. Welded connections were used with welding washers that had 3/8 in. diameter holes. Luttrell reported that the most noticeable effect of the fill was increasing the stiffness of the diaphragm system. The warping restraint provided to the deck by the insulating fill was adequate to force the failure to be at the welds as opposed to sheet instability, which had occurred in similar unfilled diaphragm tests. Expressions are presented that correlate diaphragm strength to the number of welds along the edges of the diaphragm.

Four composite diaphragm tests were performed at the University of Salford by Davies and Fisher [114]. Trapezoidal and re-entrant steel-deck profiles were used and attached to the perimeter framing members with self-drilling, self-tapping screws. Concrete cover was either approximately 2 in. or 2-3/4 in. In each case the controlling failure mode was reported as a fastener failure, with one specimen failing by a combination of fastener failure and profile collapse. The fastener spacing ranged from approximately 12 in. to 28 in. Equations were presented to determine the strength of the diaphragm based only on a fastener failure. The expressions were developed based on assumptions regarding the force distribution on the fastener patterns.

A series of nine diaphragm tests (Phase I) were performed at ISU by Porter and Greimann [390]. Numerous SDRC diaphragm parameters were varied and tested. These parameters included steel deck type, fastener type and number, and concrete thickness. Equations were developed to predict stiffness and strength of SDRC diaphragms. These equations were based on an edge zone concept, which considered the force to be transferred from the load frame into the diaphragm within a relatively narrow region adjacent to the framing members. A key component of this edge zone was considered to be the interface between the steel deck and concrete. Edge force distributions that were used to derive predictive equations were based on a linear elastic finite element analysis. Additionally, force distribution at ultimate load levels were assumed.

A series of 14 diaphragm tests were performed and reported in 1981 by ABK, a joint venture [2,3]. The ABK group is made up of members from three firms, Agbabian Associates, S. B. Barnes and Associates and Kariotis and Associates, all in the Los Angeles area. Of the 14 diaphragms only one specimen was an SDRC diaphragm. The specimen was a 20-gage deck with 2-1/2 in. of concrete cover and attached to a 20-ft. by 60-ft. test frame with intermediate framing members. Loading consisted of a series of 11 reversed cyclic, quasi-static and dynamic inputs with each series generally increasing in magnitude; however, the specimen was not loaded to complete failure. The document [2] is a data report on the experimental testing.

An additional twenty-three diaphragm tests (Phase II) were performed at ISU by Porter and Easterling [378]. Numerous SDRC diaphragm parameters were varied. These parameters included steel deck type, fasteners type and number, concrete thickness, depth-to-span ratio, loading and framing member size. As a continuation of previous research at ISU by Porter and Greimann [390], this study modified predictive equations and the means for determining the components of the equations, and inelastic force distributions were determined. A major component of the analytical portion of the study was verifying the previously assumed force distributions at ultimate. The principal focus of the analyses was to verify and define the components of the predictive equations, such that the equations might be incorporated into a design methodology. Recommendations were made regarding design parameters.

2.4 Wood Diaphragms

The quantity of general research done on wood diaphragm systems has been extensive. However, most of the diaphragm test programs to date have used static monotonic loads. In 1980, a workshop on the design of horizontal wood diaphragms [22] sponsored by the National Science Foundation and conducted by Applied Technology Council recommended: First, perform additional dynamic diaphragm tests using either cyclic loads or input from realistic earthquake motions. Second, develop mathematical models and analysis methods to predict the inelastic response of diaphragms.

Early testings of diagonally sheathed diaphragms were performed by Doyle [133-135], Atherton and Johnson [36], Stillinger et. al. [462], Stillinger [459,460], Johnson [263,265], Currier [106] and Burrows and Johnson [73,268]. Several series of 20x60 foot diaphragm parameters were varied and tested. These parameters included combinations of lumber sheathing, framing, and fasteners. Testing performed by Stillinger [460] has shown that diagonally sheathed diaphragms can sustain ultimate loads up to about 1600 lb. per foot. An extensive review of wood diaphragm testing was done by Elliott [149].

Full-scale plywood-sheathed diaphragm tests were performed at Oregon Forest Products Laboratory by Stillinger and Countryman [461], and Johnson [264,266]. The same type of tests were performed by Countryman [101], Countryman and Colbenson [102], and Tissell [478] at American Plywood Association Laboratory. A review of these tests shows that the behavior of wood diaphragms is highly nonlinear. Most of these tests were, however, noncyclic.

An investigation of the structural damping characteristics of composite wood structures subjected to cyclic loading was conducted by Young and Medearis [518] at Stanford University. A series of eight 8x16 ft. plywood shear wall tests were performed with varied parameters. These parameters included plywood thickness, perimeter nailing, re-nailing, and pre-cycled diaphragms. As part of their conclusions, Young and Medearis stated the following: First, considerable frictional damping was generated at the common surface of the plywood sheets, as well as between the plywood and the framing members. Second, none of the panels experienced any sudden failures; and, although most of them were loaded until a number of different failure locations were observed, most of which were quite local in nature. Third, the energy absorption properties of the panels were very great, with about 60% of the input energy being absorbed during any given half-cycle. From their theoretical work using the test results, values of an equivalent viscous damping ratio were found to average 0.10 for shear walls with plywood on both sides.

Ewing et. al. [158], and Luttrell et. al. [186], have been working in development of mathematical models for static and dynamic analysis of wood diaphragms to predict their inelastic response when subjected to time variant forces. A lumped parameter analytical model was used by Ewing for analyzing wood diaphragms supported on masonry walls. The analysis provided quantitative estimates of the out-of-plane excitations induced in the masonry walls. The analytical model included a nonlinear hysteretic element that simulates the behavior observed in cyclic tests of wood diaphragms. The results of the nonlinear dynamic analysis were compared with corresponding results obtained with linear elastic and nonlinear elastic assumptions. The nonlinear elastic case showed a general reduction (relative to the linear elastic case) in peak responses, and the nonlinear hysteretic case showed further significant reductions in all response quantities. As part of their work they stated that elastic analyses overestimate the actual response and that consideration of the hysteretic characteristics of the wood diaphragms is necessary for more realistic response predictions.

Research made by Luttrell et. al. [186] has been toward the establishing of a method for predicting joint slip and its relationship to overall shear displacements. The aim of their work was toward developing a dynamic performance model and related exploratory physical testing. Full scale models were tested to determine stiffness and slip with large groups of nails, and associated small scale tests for nail slip tendencies; and preliminary stiffness models were developed. As part of their comments, they stated that for extension to the dynamic model, the key appears to be in the damping characteristics associated with joint slip. To that end, many separate tests have been done to quantify joint slip characteristics.

A series of 14 diaphragm tests were performed and reported by ABK, a joint venture [2,3]. Of the 14 diaphragms, eleven specimens were wood diaphragms. The wood diaphragms were 20x60 foot, three with straight sheathing, three with diagonal sheathing, and five with plywood, and included double board systems. As part of their results, they showed the values of the basic characteristics for a nonlinear hysteretic spring model, associated with wood diaphragms tested. (The document [2] is a data report on the experimental testing).

2.5 Precast Concrete Diaphragms

During previous seismic events, the performance of precast concrete units without topping has been poor, while the precast concrete units with topping have exhibited variable to good performance [150]. Martin and Korkosz stated that the absence of continuity and redundancy (between the precast slabs) has caused some designers to question the stability (of precast structures) under high lateral loads [314].

An experimental investigation of the shear diaphragm capacity was undertaken by Concrete Technology Corporation in February 1972 [93]. Three 15 ft. long, 4 ft. wide, 8 in. thick Spiroll Corefloor hollow-core slabs were assembled side by side. The two longitudinal joints were filled with nonshrink grout. At the ends of the plank system two end beams 4 in. x 8 in. x 12 ft. each were cast in place. Two #3 grade 60 deformed bars were placed in each beam. The objectives of this test were to measure and evaluate the ability of 8-inch Spiroll Corefloor slabs to transfer horizontal shear through the grouted longitudinal joints without shear keys, as well as to determine the coefficient of friction, which served as a direct measure of the effectiveness of shear friction reinforcement in the end beams. The longitudinal joints were subjected to pure shear as the load was applied to the center slab while the exterior slabs were held in place. The shear strength was not tested to ultimate capacity, since a measure of the shear friction effectiveness was one of the desired objectives. After the joints were artificially cracked, the coefficient of friction was measured and was found to vary between 1.3 and 2.0. These values indicated that the reinforcement had performed satisfactorily and that the 1.0 value was conservative for planks with extruded edges.

A publication of the Concrete Technology Associates by Cosper, et. al. [98] reviewed hollow-core diaphragm test results for the shear strength of the grouted keyway between adjacent 12-inch Dy-Core panels. The deck specimen consisted of four 22 ft. long by 12 in. thick DY-CORE planks. The three longitudinal joints were filled with shrinkage compensating grout. The edge beam at the end of the planks measured 9 in. x 12 in. x 16 ft. long and were cast in place. Reinforcement in the edge beams included six #5 longitudinal bars, four of which were anchored into the end core a distance of 24 in. Number four grade 60 U-bars were anchored around these four rods and into the core a distance of 24 in. An L-shaped #4 bar held the two bottom longitudinal #5 bars in position. Longitudinal shear loading was accomplished by applying a load against sixteen 1/2-inch prestressing strands, which were in an "x" arrangement across the seam. Parameters researched included the following: 1) the shear capacity of an uncracked grouted joint, 2) the effectiveness of shear friction reinforcement in transferring shear across a joint, 3) the ductility of the system after the bond between panels had fractured, and 4) the effect of cyclic loading on the system. The uncracked grouted seam demonstrated a high capacity in resisting lateral shear loads. Shear friction steel placed in the edge beam supplied adequate clamping forces once the seam had fractured. Ductility demands were satisfied as well, since the shear strength continued to increase after joint displacement. Finally, the diaphragm exhibited sufficient resistance to cyclic loading by maintaining a stabilized strength after repeated cycles above design requirements.

Another experimental study, by Reinhardt and Hartjes [411], tested the joint between hollow-core planks under shear loading while subjected simultaneously to a normal force. Variable strengths of mortar and lengths of the grouted connection (0.3 to 2.1 meters) were accommodated for the single seam. Three different mortars were tested. The Lime-Cement mortar reached a maximum shear stress of 0.4 N/mm2 in a joint of 0.3 m length and also of the 2.0 m length. Clearly the strength of this very low-quality mortar did not depend upon the absolute length of the joint. The other two mortars showed higher shear stresses and an influence exercised by the joint length. The shorter the joint the higher was the maximum average shear stress. With better mortar quality the length influence became stronger. Joint length was found to have a significant influence upon shear stress at fracture for their particular testing configuration. Failures were characterized by brittle fractures of the bond at the mortar and grout interface.

Each of these tests used a slightly different testing frame. With such testing arrangements, however, the actual maximum shear is not simply the load divided by the contact area. A correction factor which accounts for the non-uniformity of the shear stresses must be used. Chow, Conway, and Winter state that the distribution of shear stresses in deep beams depart radically from that given by the ordinary, simple formulas [88]. Using finite difference, strain-gage measurements, and photoelastic measurements, Roarke and Young have tabulated the correction factor for various testing arrangements [416].

2.6 Reinforced Concrete Diaphragms

Lateral forces, typically produced by earthquakes or winds, are resisted by the use of a space frame system and/or shear walls. In either case, the lateral loads are transmitted from one wall to another through the floor system. The distribution of horizontal forces to the shear walls or space frame system depends on the properties of the diaphragm slab and the resisting system. The required diaphragm stiffness varies from one structural configuration to another. In-plane flexibility can be beneficial in a long roof supported on low, uncoupled concrete perimeter walls, due to dynamic attenuation of lateral ground accelerations. On the other hand, to improve the lateral force resistance of a multistory structure, it is desirable for the in-plane stiffness of floor and roof diaphragms to exceed the stiffness of the vertical subsystems to which they are connected [90].

Nakashima, Huang, and Lu [332], reported in 1982 an experimental work done at Lehigh University. This research considered reinforced concrete floor slab systems with edge beams subjected to in-plane loads. The main objectives of their experimental study were to examine the strength, stiffness, and deformation characteristics of sample units of the floor slab system and to provide a basis for comparison with an analytical model. The testing program included three types of tests: the stiffness test, the strength test, and the repaired strength test. In the stiffness test, each specimen was tested as a whole unit to determine the elastic in-plane stiffness characteristics of the slab panels. In the strength test, each slab panel of the specimen was tested individually as a cantilever to examine its stiffness, strength and deformation characteristics in both the elastic and postelastic ranges. After the strength test the slab panel was repaired using an epoxy injection technique; it was then retested under the same loading conditions. A nonlinear finite element program was developed to analytically investigate the in-plane characteristics of floor slabs. As part of their conclusions, they stated the following: First, the elastic finite element analysis reasonably predicted the elastic in-plane stiffness of the test panels. Second, in all strength tests, the development of the major crack controlled the in-plane ultimate load. Cyclic loading or the application of the design service vertical load reduced the ultimate load by 15 to 25 percent. Third, stiffness of the test panels continuously degrade as load increased. Fourth, repair by the epoxy injection technique restored the test panels to their earlier strength, but the repaired panels were consistently less stiff than the original panels. Finally, the nonlinear finite element analysis successfully predicted the in-plane strength of the test panels and reasonably duplicated the experimental load-deflection curves and crack patterns.

2.7 Hysteretic Models

A complete description of the behavioral characteristics of a structure throughout the plastic and elastic ranges can be obtained with a hysteretic model. This type of model predicts the force-displacement relation for a system utilizing stiffness and strength information. Riddell and Newmark have established a set of desirable characteristics for a hysteretic model. These features can be summarized as follows [413]:

1. Reality - The selection of model parameters that are directly associated with known physical characteristics.
2. Accuracy - The model must portray the measured results as closely as possible.
3. Simplicity - The prediction should be completed with the simplest method possible.
4. Consistency - The relationship between the response variable and any specific parameter should be consistent.

Many hysteretic models have been developed in the past. Each successive model has improved upon the first effort in some way. The characteristics of any reinforced or prestressed concrete model have become more refined and can be briefly stated as follows [354]:

1. The stiffness must change with the cracking of the concrete.
2. The loading stiffness in the second cycle is lower than the that of the first, and the load at peak displacement is nearly the same strength.
3. The average peak-to-peak stiffness decreases with the increase of the maximum displacement amplitude.
4. There is a tendency for very low incremental stiffness near the origin followed by a stiffening region (pinching).
5. Prestressed concrete is represented by a softening model.
6. The hysteretic curve for a prestressed structure has relatively small residual curvature.

The simplest type of hysteretic model is the bilinear model. A special case of this type of model is the elasto-plastic system. Many versions of the bilinear model have been developed; however, this type of model provides only a rough estimate. It contains no degradation properties and has an unrealistic transition point in the skeleton curve (a curve which describes the force-displacement relation obtained by increasing the magnitude of the load acting on the structure).

Jennings developed a general nonlinear hysteretic force displacement relation in the early 1960's which utilized closed form mathematical formulas [256]. The skeleton curve used a formula similar to that first proposed by Ramberg and Osgood [408] to describe relations between stress and strain. This model employed smooth rounded curves which were general enough to describe the behavior of systems ranging from linear to elasto-plastic. The skeleton curve utilized the following relation:

$$x / x_y = p / p_y + \alpha (p / p_y)^r \tag{2-1}$$

where:

- x = the displacement of the structure, in.
- x_y = the calculated displacement, in.
- p = the restoring force, kips.
- p_y = the yield force, kips.
- α, r = empirical constants.

The ascending and descending branches of the hysteresis loop were defined respectively by:

$$(x+x_o) / 2x_y = (p+p_o) / 2p_y + \alpha[(p+p_o) / 2p_y]^r \tag{2-2}$$

$$(x-x_o) / 2x_y = (p+p_o) / 2p_y + \alpha[(p+p_o) / 2p_y]^r \tag{2-3}$$

where all the variables are the same as above and

- x_o = the displacement at the point where the loading is reversed, in.
- p_o = the load at the point where the loading is reversed, kips.

Degrading-type models introduced the effect of stiffness degradation caused by load reversals in inelastic ranges. Clough and Johnston [92] in 1966 proposed a degrading bilinear model, improving the elasto-plastic model by accounting for the stiffness degradation observed during the cyclic loading of reinforced concrete components.

Iwan [250] developed a similar model. This model was slightly more general than the previous one. It employed four types of behavior: elastic, elasto-plastic behavior, simple coulomb slip behavior, and irreversible or directional behavior. A complex set of definitions and integrations compose the major elements in this model.

The next type of model was developed by Takeda in the early 1970's and stands as one of the founding models. The Takeda model is based on experimental results and operates on a trilinear skeleton curve [471]. This primary curve is composed of segments connecting the origin, cracking point, yielding point, and ultimate point. The model contains many rules which govern the slopes of all the components. These guidelines mandate degradation, but not pinching action.

In the late 1970's, another model was developed to simplify the rules associated with the Takeda model and to account for the effects of pinching. This model, which is known as the Sina Hysteresis Model [431], contains a skeleton curve made up of three parts and is defined by nine rules. An example of this type of model can be seen in Figure 2 and the rules for this model are summarized in Figure 3.

Another modified version of the Takeda Model is the Otani Hysteresis Model [355]. This model was originally used to represent the stiffness variation of a joint spring in conjuntion with a flexural spring. The primary curve in this system is bilinear because the breakpoint is at yielding. Eleven rules are associated with this model, making it somewhat more complicated.

The final type of model employs many of the characteristics associated with the techniques previously described. This model is part of the Lumped Parameter Model Program, LPM, and is named the EKEH model (element #11) [158,159]. The Technical Coordinating Committee on Masonry Research (TCCMAR) has requested that this model be considered as part of the overall project. It has nonlinear, inelastic, degrading and pinching capabilities as is shown in Figure 4. The skeleton curve consists of a second order

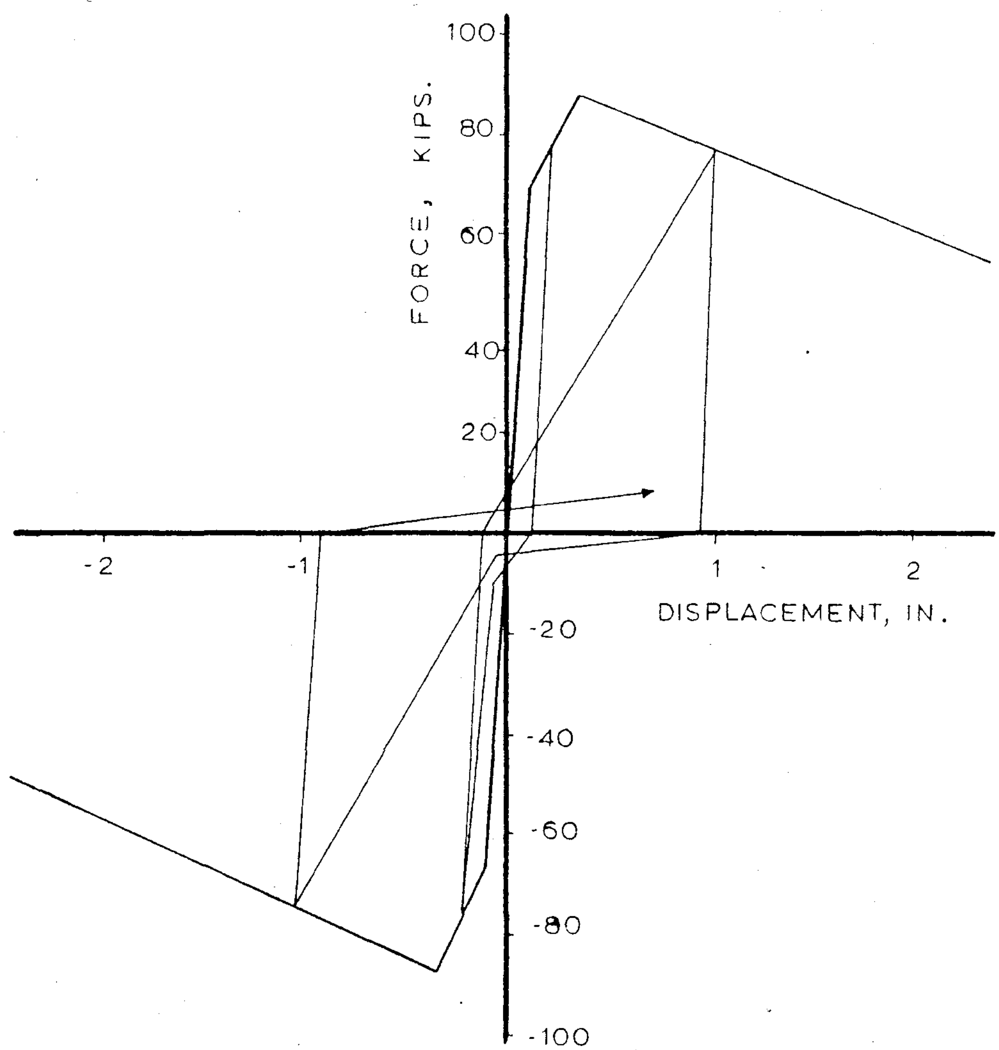

Figure 2. Example of several cycles of hysteretic curve for ISU Test #5 employing modified Takeda model (Sina model) [376]

INITIAL LOADING	$F(p) \leq F(c)$ Follow initial slope of skeleton curve $F(p) \geq F(c)$ Follow slope of skeleton curve from FME to yield
CURRENT POINT ON FME TO YIELD BRANCH OF SKELETON CURVE	Loading - Follow skeleton curve from FME to yield Unloading - Unload at slope of $S_1 = C_y * (\frac{D_y}{Dm})^{0.5}$
CURRENT POINT ON SKELETON CURVE FROM ULTIMATE TO YIELD	Loading - Follow skeleton curve from ultimate to yield Unloading - Unload at slope of $S_1 = C_y * (\frac{D_y}{Dm})^{0.5}$
LOAD REVERSAL	If not previously yielded, connect the intersection of the x-axis to the yield point If previously yielded, connect intersection of x-axis and ultimate point
ADDITIONAL POINTS	See Reference 431 for additional rules on unloading, loading, and load reversals.

Figure 3. Rules for modified Takeda model [431]

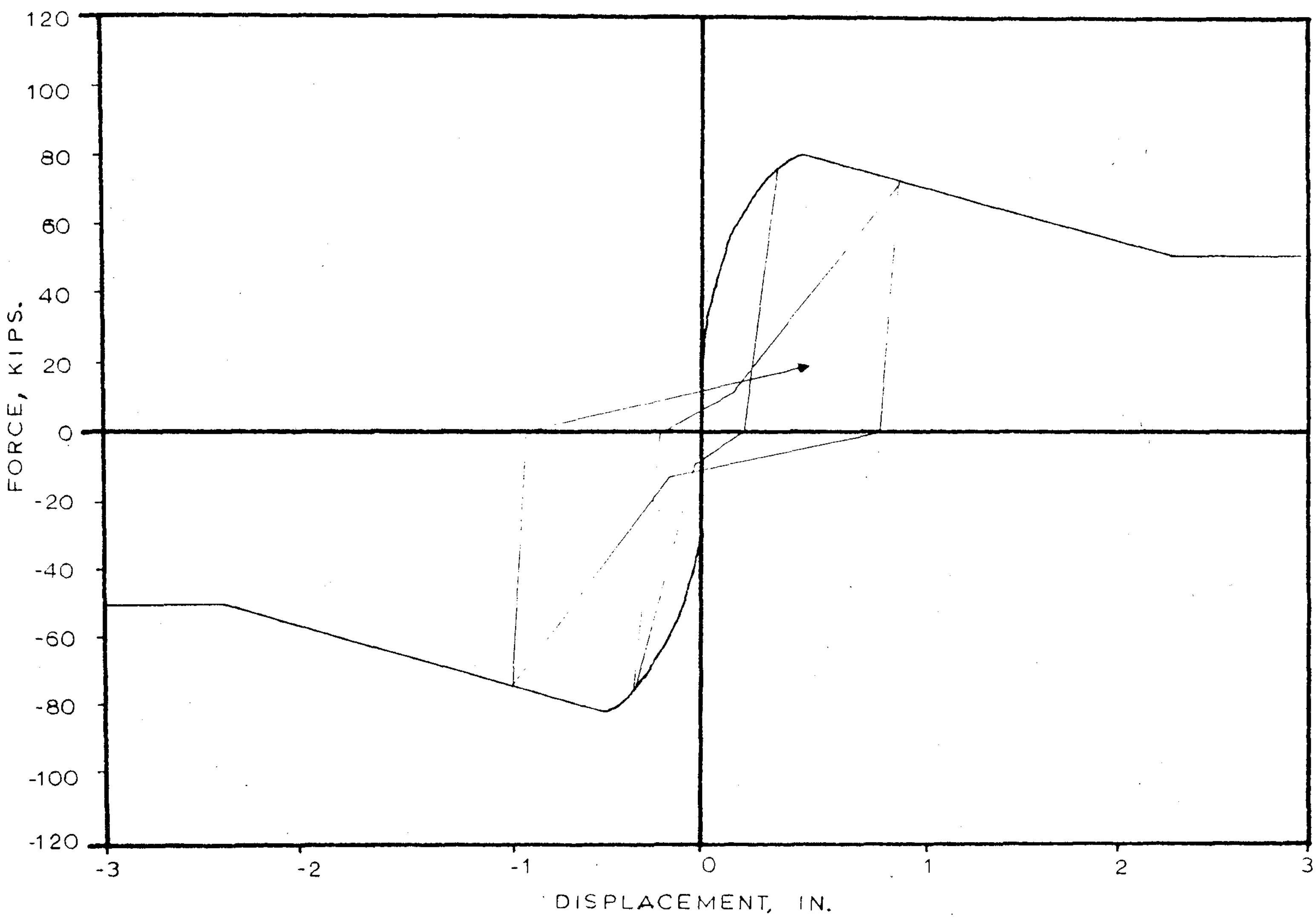

Figure 4. Example of several cycles of hysteretic curve for ISU Test #5 employing EKEH/LPM model [376]

function and two linear segments with the following format:

$$F(e) = F_p * e / (F_p / k_i + \alpha|e|) \qquad -e_p < e < e_p$$

$$F(e) = F_p[(e_m - |e|)/(e_m - e_p)]\text{sign}(e) \qquad e_p < |e| < e_t \qquad (2\text{-}4)$$

$$F(e) = \beta F_p \text{ sign}(e) \qquad -e_t > e > e_t$$

where

- $F(e)$ = spring deformation force, kips.
- e = spring deformation, in.
- e_p = deformation at peak strength, in.
- e_m = intersection of the post peak strength envelope line with the deformation axis (e Axis)
- β = post peak strength lower limit factor.
- F_p = peak strength, kips.
- k_i = initial stiffness, kips/in.

$$e_t = e_m - \beta (e_m - e_p) \qquad (2\text{-}5)$$

$$\alpha = 1 - F_p/k_i e_p \qquad (2\text{-}6)$$

The post peak strength factor, ß, can be defined as the load after the usable strength has been achieved. The ß may be determined for each limit state mode. Unloading occurs along a degrading stiffness slope defined by:

$$K_u = K_i (F_p / K_i * e_{max})^{\gamma} \qquad (2\text{-}7)$$

where

- K_u = the degraded stiffness slope, kips/in.
- γ = a degradation stiffness constant.
- e_{max} = maximum displacement reached in all previous cycles.

The latter constant, γ, is defined from experimental test results and acceptable values range between 0.5 and 0.8. Values above 0.8 lead to nonconservative hysteresis loops. Skeleton curve for the EKEH/LPM model is shown in Figure 5.

Reloading occurs along a path controlled by the maximum force and the current pinch force. (This force is defined as in the Sina model). It starts from the point of zero force and continues through the pinch force until it intersects the straight line connecting the origin and point"a" (see description below). At this intersection, it follows the latter line segment until it intersects point "a" on the skeleton curve. The location of point "a" is defined by one of the following conditions:

1. If the peak strength has not been exceeded in all previous cycles, point "a" is defined as the point on the envelope that corresponds to the maximum displacement.

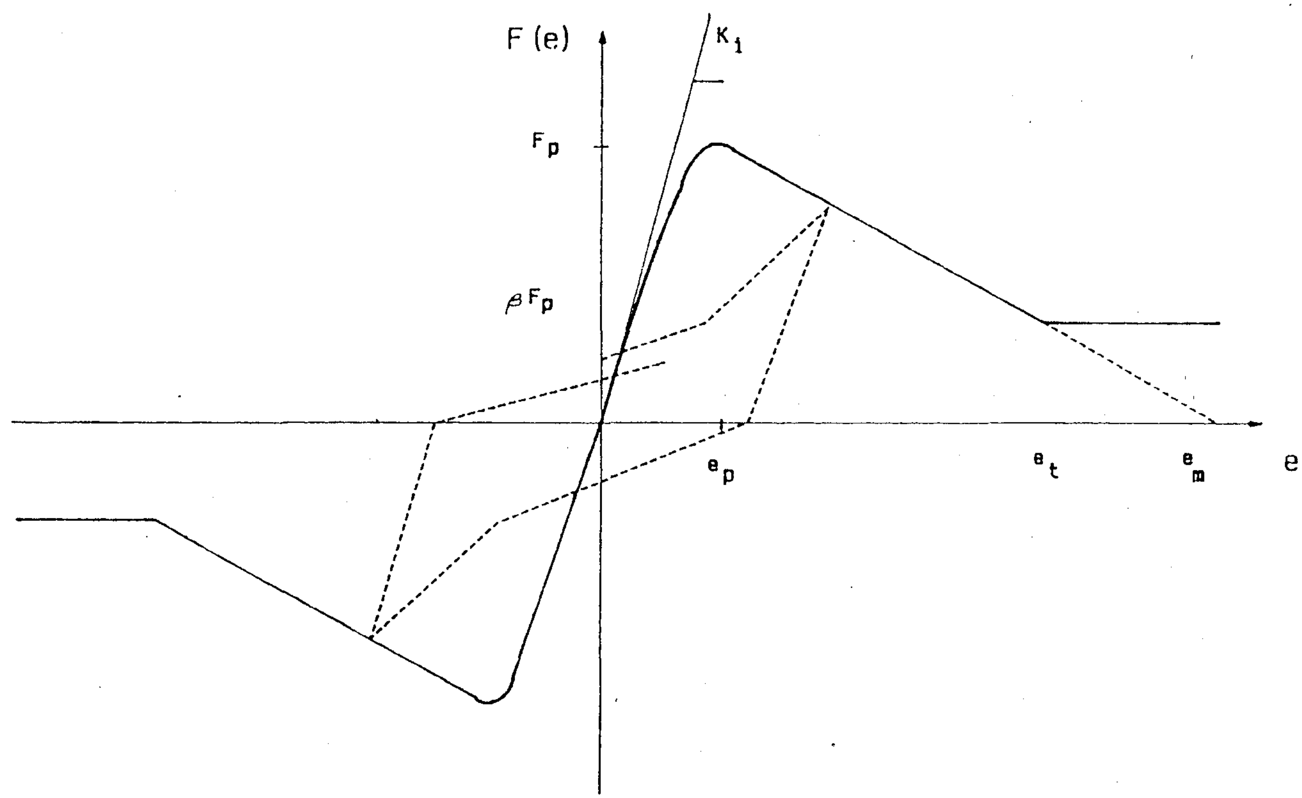

Figure 5. **EKEH model skeleton curve** [376]

2. If the peak strength has been exceeded, point "a" is defined by the deformation equal to the maximum deformation and a force equal to 0.8 times the force on the envelope curve that corresponds to the maximum deformation.

In other areas, loading and unloading follow the same basic rules developed in the models described earlier.

Many hysteretic models have been developed over time. Improvements in concrete models have been made to allow for stiffness degradation and pinching action. The Takeda model, and it subsequent improved versions, as well as the EKEH/LPM model represent the most comprehensive models currently in use. These models are in concordance with the four desirable characteristics established by Ridell and Newmark [413]. Experimental data associated with the EKEH/LPM model are given in the Appendix A, Section 8.3, Table A8, and Section 8.4, Table A9.

Additional work on the development of a hysteretic model for hollow-core plank diaphragms is currently being done at Iowa State University as part of TCCMAR Research program (Task 2.4a) [348].

3. DATA FORMULATION OF PREVIOUS WORK

3.1 General

This chapter addresses a detailed description of the experimental program done in diaphragms including the test rationale, brief description of the specimens, test setup, instrumentation, hydraulic actuation system, and test data acquisition for the following systems:

	DIAPHRAGM SYSTEM		REFERENCE
1.	Cold-formed Steel..................	a)	Luttrell, et. al. [148,308]
		b)	Lukens [299]
2.	Wood................................	a)	Young and Medearis [518]
3.	Steel-Deck-Reinforced Concrete.....	a)	Porter and Easterling [378]
4.	Wood, Cold-Formed Steel and Steel-Deck-Reinforced Concrete.....	a)	ABK, A Joint Venture [2,3]
5.	Precast Prestressed Concrete.......	a)	Porter, et. al. [376]

3.2 Research performed by Luttrell, et. al. [148,308]

The following discussion of data formulation on cold-formed steel diaphragms was partially taken from Reference [148]. Experimental data result of this research is included in the Appendix A, Section 8.1.1, Tables A1 and A2 of this report. A series of 160 tests were performed and reported by Luttrell and Ellifritt at West Virginia University as a result of research on three general types of steel deck under various conditions of fastener arrangement, purlin spacing, gage, and material yield strength. This study was part of the support document for the Steel Deck Institute Diaphragm Design Manual [306,307]. Two major behavioral parameters were obtained, ultimate strength and shear stiffness. Ultimate strength, S_u, designates the total jacking force required to produce failure in a diaphragm divided by the length of the diaphragm in the direction of the applied load. Shear stiffness, G', is a measure of the relationship between in-plane load and the deflection in the direction of that load. The shear stiffness is defined in Figure 6.

Three types of deck were considered: narrow rib ("A" deck), wide rib ("B" deck), and intermediate rib deck. Among the wide rib decks tested, there were two variations in the side lap arrangement: the standing seam side lap ("WB"), and the flat side lap ("W"). Tests were made on 16, 18, 20, and 22 gage decks with

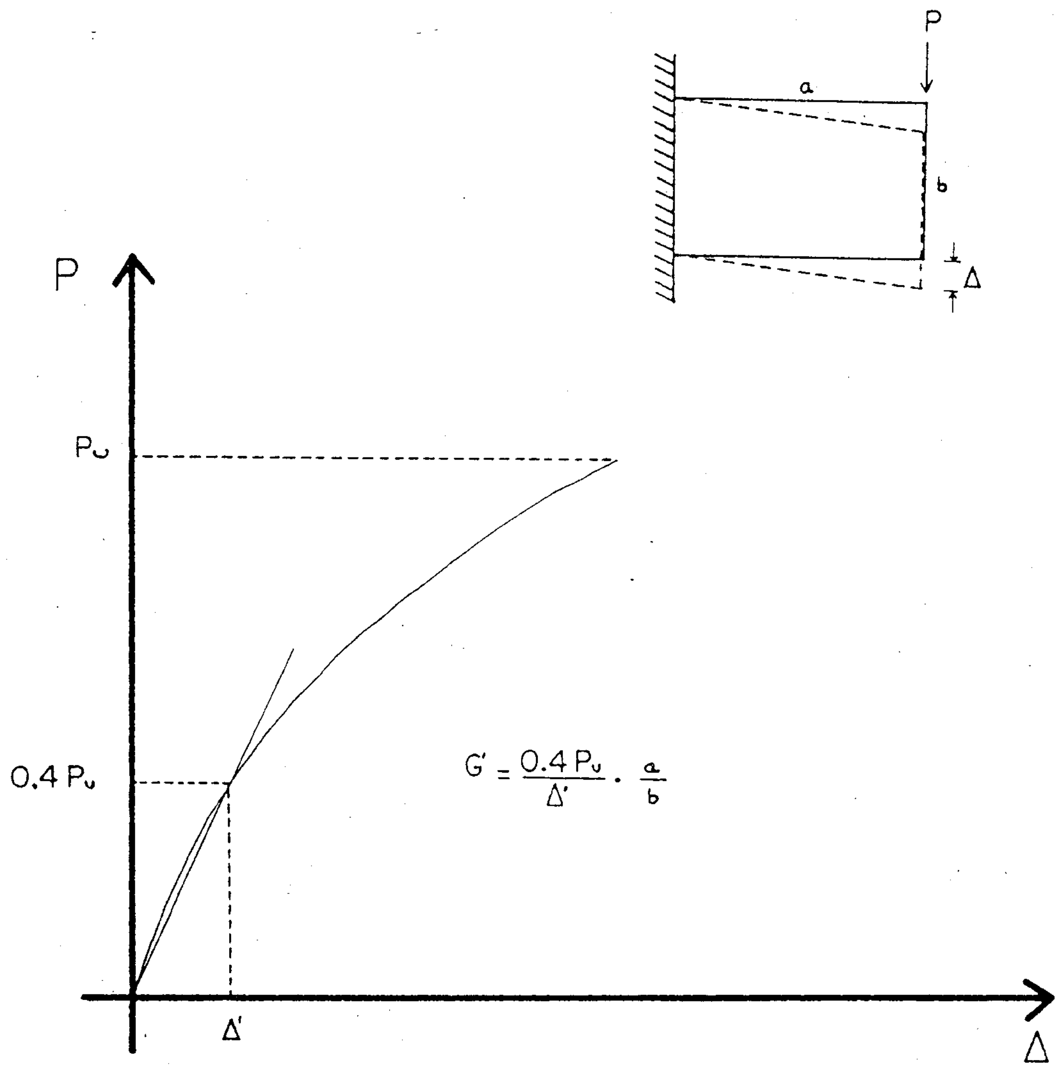

Figure 6. Definition of shear stiffness G' [148]

lengths of 12, 16, and 20 feet. Panel widths tested were 18, 24, 30, and 36 inches. All tests were made on a horizontal cantilever test frame. The connections between the perimeter members of the frame were made with light clip angles and considered as pinned. The entire frame was supported on rollers to eliminate the possibility of developing frictional resistance during deformation. Purlins were fastened to the frame with pinned connections and spacing was variable. Welds were made with E6013 1/8" diameter electrodes with sufficient heat for fusion. Various weld arrangements were used.

The loading apparatus for all tests consisted of one hydraulic jack and load cell arrangement in line with the center line of the south edge member at the southwest corner. A tensile load was applied by means of a high strength rod threaded through the reaction frame and connected to the edge beam at a level where the diaphragm attaches to the frame. Load was applied in increments from zero to failure with deflection measurements made at each stage of loading. Deflections were measured with Ames dial gages accurate to 0.001" at all corners in the plane of the diaphragm as shown in Figure 7. From these measurements, the true diaphragm deflection was calculated according to the formula,

$$\Delta = \Delta_4 - [\Delta_2 + (\Delta_1 + \Delta_5)\ a/b] \qquad (3\text{-}1)$$

where Δ_1, Δ_2, Δ_4, and Δ_5 are measured movements at the corners in inches. In each test, the diaphragm was loaded to failure, which was initiated in a variety of ways. Weld failure was generally because of the sheet tearing away from the weld. Although unusual, welds sometimes separated from the perimeter beam while still attached to the sheet. If the lip on the male rib is small, as in the narrow rib decks, sudden local buckling of the lip leads to overall buckling of the flute. If the return on the male rib is large, as in the wide rib decks, the flute will generally fail as a slender compression strut. In many of the diaphragms tested, weld failure and buckling were very closed allied and an assignment of precedence to one failure over the other was difficult. In general, all the modes of failure were present to some degree. The failure of a weld or the buckling of a flute did not necessarily mean that the ultimate diaphragm load had been reached. However, the additional increase in load after an initial failure of this type usually did not exceed ten percent.

3.3 Research performed by Lukens [299]

The following discussion of data formulation on cold-formed steel on wood frame diaphragms was taken from Reference [299]. Experimental data result of this research is included in the Appendix A, Section 8.1.2, Table A3 of this report. Twenty-two full-scale cantilever tests on steel clad, timber-framed diaphragms were performed at Iowa State University by Lukens. Ultimate

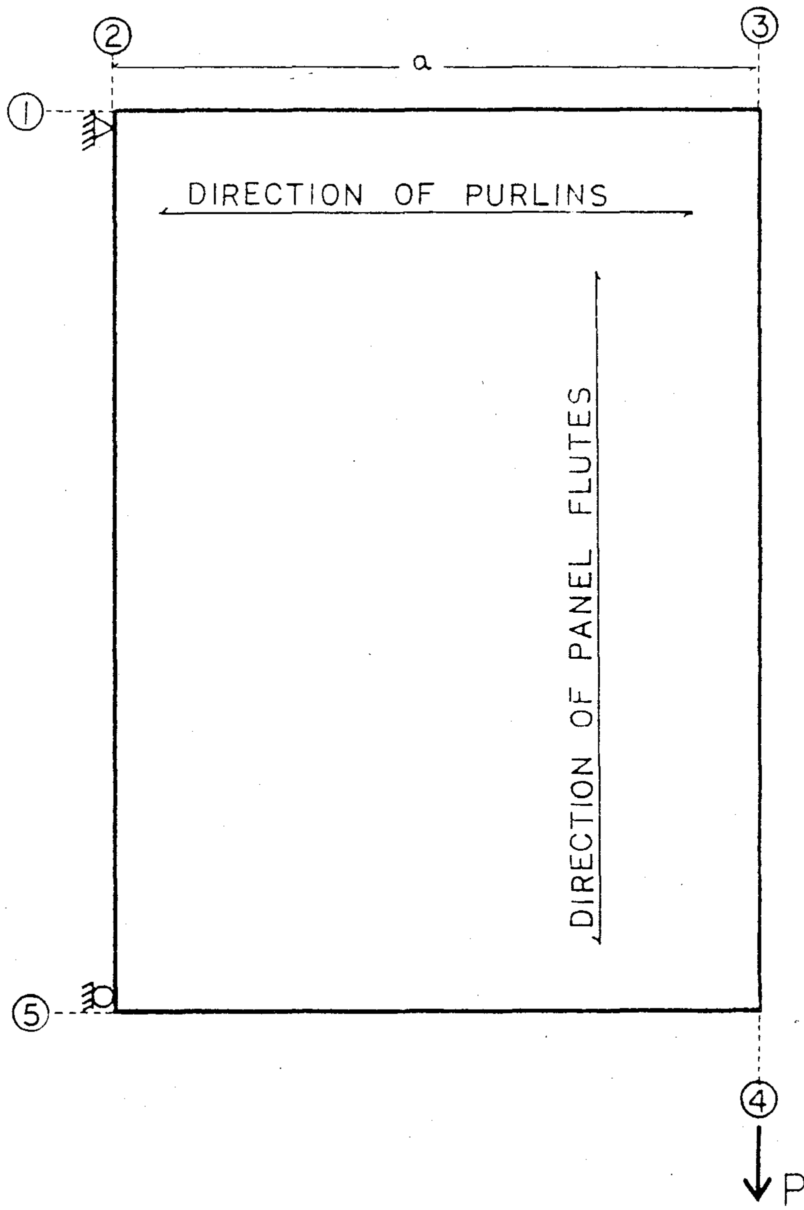

Figure 7. Schematic of diaphragm test showing location of deflection gauges [148]

strength, S_u, and shear stiffness, c, were evaluated. Variables included reverse loading, types of wood for purlins, purlin to chord connections, stitch screws and length. Ultimate strength, S_u, is defined as the ultimate load at which a panel will not carry any additional load for a given period of time (approximately two to three minutes). The shear stiffness, c, is defined in Figure 8.

Each panel was constructed with three sheets of Grandrib 3, manufactured by Fabral, Lancaster, Pennsylvania. These sheets were 29 gage galvanized steel with a coated thickness of 0.0172 in. and base metal thickness of 0.0135 in. The minimum yield strength of the base metal was 80 ksi. The sheets spanned the length of the panel. In all panels except Panel 13, sheets were fastened in valleys with 1.0 in. long, #10 wood-grip, self-tapping screws. The screws at the seams of the sheets were fastened only through one sheet. In addition to these screws, panels 20-22 were fastened at the seams of the sheets with 0.5 in. long, #8 self tapping stitch screws. These stitch screws were spaced approximately 12 in. on center through the ribs of the two sheets at the seams. Panel 13 was fastened with 16d threaded galvanized steel nails. Nails were hammered through the ribs.

The timber-frames were constructed with 2x8 in. top chords and 2x4 in. purlins as shown in the Figure 9. The chords for Panels 1-4 were #2 and better spruce-pine-fir, and the chords for panels 5-22 were #2 and better Douglas fir-larch. The purlins for Panels 1-9 were standard and better spruce-pin-fir, and the purlins for Panels 10-22 were standard and better Douglas fir-larch. In all the panels, except Panels 2 and 3, the purlins were nailed on edge to the chords with one 60d spike and 10d toenails. The purlins for Panels 2 and 3 were nailed on edge to the chords with one 60d spike at all location plus one Simpson H-1 anchor nailed to the purlin and the chord at left corners C and D. The length of each panel was measured from the outside faces between the first and last purlin. The different lengths for the panels were 6, 8, 12, 16 and 20 ft. with each length having the purlins spaced equally at 23.5, 23.625, 23.75, 23.813 and 23.85 in. on center, respectively. The width, a, of the panels was the distance between the centerline of the two chords. All panels had a width of 9 ft.

A cantilever testing procedure was used to calculate shear stiffness and strengths. All panels, except Panels 2, 3, and 13 were tested twice (Test A and Test B). During Test A, panels were loaded from zero to failure, which was a split in the chord at corner C and D caused by edge purlins pulling or pushing across the chord. After Test A was completed, four 3x3x3/2 in. angles 1.5 in. long were screwed into the chord and the purlin at corners C and D. These angles repaired and strengthened the purlin to chord connection so that it would not fail during Test B. In Test B, this strengthened panel was loaded beyond the failure of Test A to a new failure.

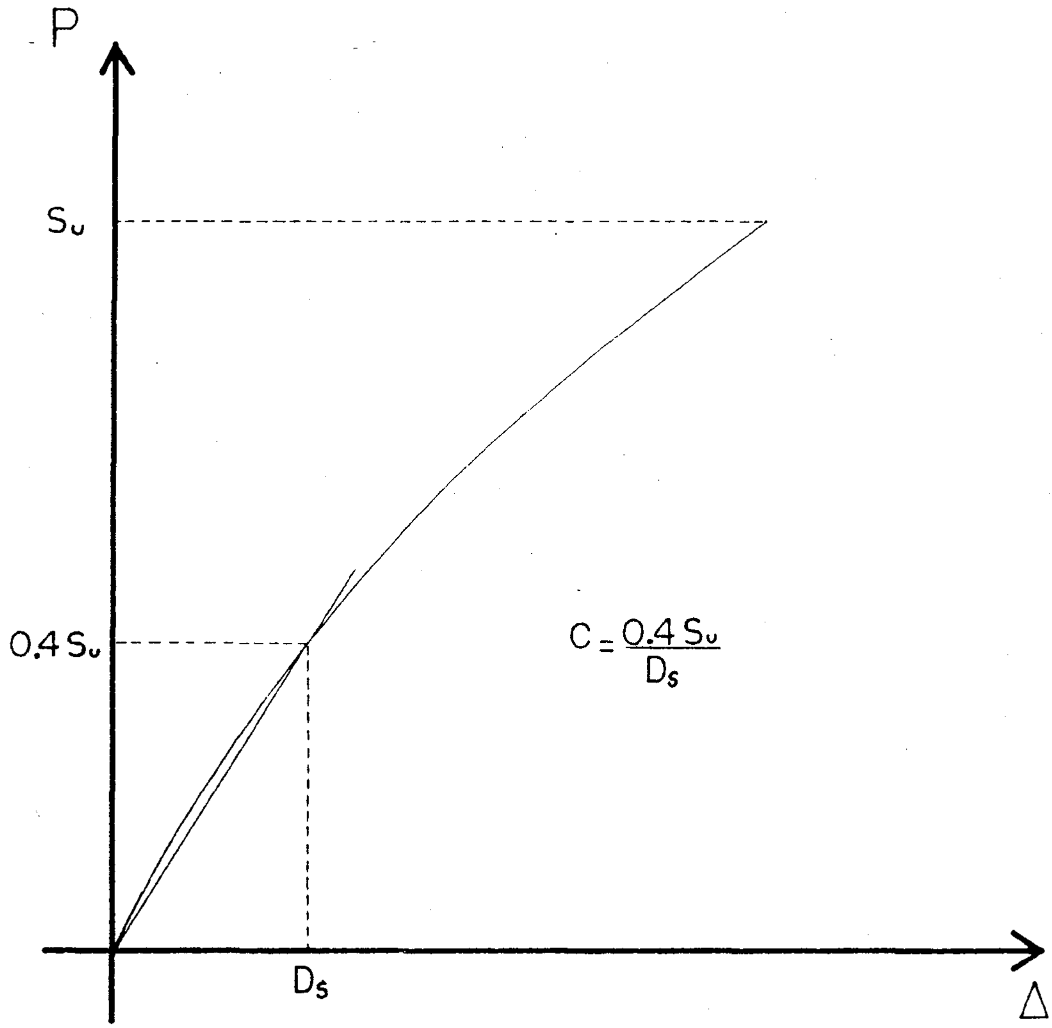

Figure 8. Definition of shear stiffness C [299]

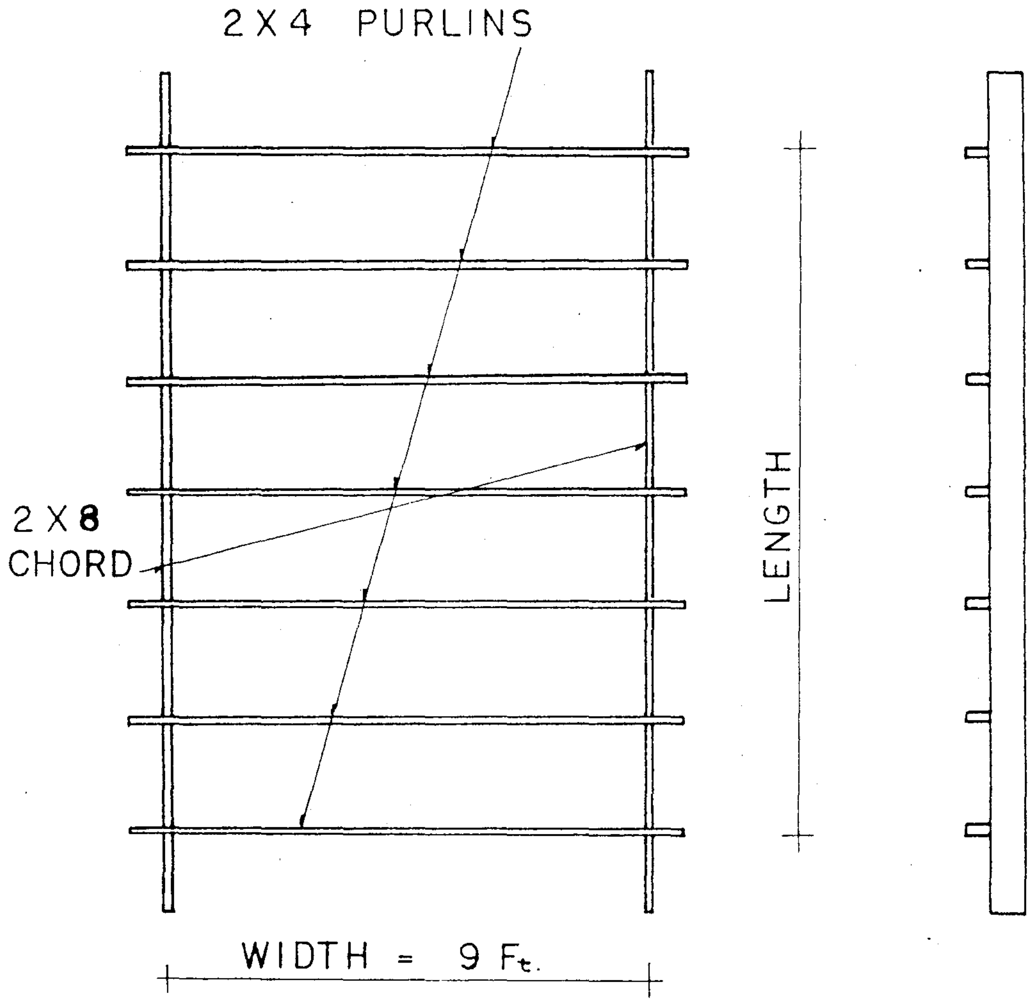

Figure 9. Typical timber framing [299]

Load, P, was applied to the chord at Corner A with a turnbuckle and recorded from a 5000 lb. capacity dynometer, except for Panel 20 Test B. The load for Panel 20 Test B was recorded from a 10,000 lb. capacity load cell. For Test A with lengths 12 ft. or greater the load was generally applied in 100 lb. increments to failure with the load released every 500 lb. to record permanent deflection. For lengths less than 12 ft. the load was generally applied in 50 lb. increments to failure with the load released every 250 lb. to record permanent deflection. For Test B the load was applied in about 500 lb. increments to the Test A failure load. Then the load was applied in 100 lb. increments to failure for lengths less than 12 ft. and in 200-300 lb. increments to failure for lengths 12 ft. and greater. Deflections were recorded every load increment.

Panels 6-9 were subjected to reversing loads. Reverse loading was started by pulling the panels at Corner B in 100 or 200 lb. increments to 1100 lb. for Panels 6, 7 and 8 and to 700 lb. for Panel 9. The 1100 lb. and 700 lb. loads were approximately equal to 40% of the strength of a 12 ft. panel with spruce-pine-fir purlins for Test B and A, respectively. Then the load at Corner B was released to zero and applied to Corner A in the same manner. After a given number of cycles, load was applied at Corner A to failure. Panel 6 had 10 cycles, Panel 7 had 9 cycles, Panel 8 had 20 cycles, and Panel 9 had 20 cycles. This load history completed Test A. Test B loaded the same as the non-reverse loading panels.

The cantilever test apparatus is shown in Figure 10. The chord at Corners C and D was fixed to the floor and the chord between Corners A and B was placed on rollers. Deflections were recorded from dial deflection gages.

The shear deflection, δ_s, was calculated with the following equations:

$$\delta_T = \delta_1 - \delta_2 - (\delta_3 + \delta_4)\ a/b \qquad (3\text{-}2)$$

$$\delta_B = Pa^3/3E_pI \qquad (3\text{-}3)$$

$$\delta_S = \delta_T - \delta_B \qquad (3\text{-}4)$$

where

δ_T = net total deflection at Corner A, in.
δ_B = bending deflection at Corner A, in.
δ_1, δ_2, δ_3, δ_4 = deflection gauge reading at corners, in.
P = load applied to panel, lb.
E_p = modulus of elasticity of edge purlins, psi
I = second moment of inertia of edge purlins about centerline of panel, in^4.

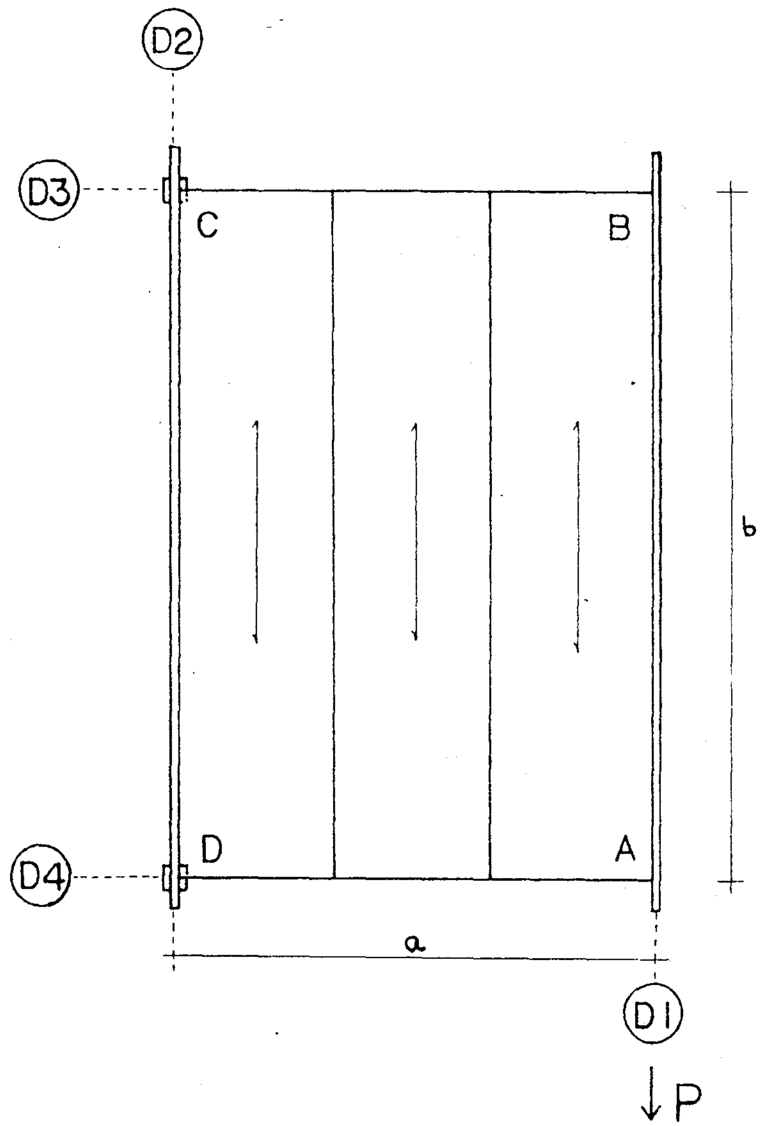

Figure 10. Cantilever test apparatus [299]

3.4 Research performed by Young and Medearis [518]

The following discussion of data formulation on wood shear walls was taken from Reference [518]. Experimental data result of this research is included in the Appendix A, Section 8.2.1, Table A6 of this report. A series of eight plywood shear wall tests were performed by Young and Medearis as part of their work on the structural damping characteristics of composite wood structures subjected to cyclic loading. Variables included plywood thickness, precycling, nailing, and re-nailing.

Each 8x16 ft. panel tested consisted of two 8x8 ft. wood walls joined side by side by 3-5/8 in. machine bolts at 3 ft. c-c. All tests were made on a vertical test frame. The connections between the panel and the steel frame was made with one 3x3x1/2 in seat angle with 3/4 in. stiffener plates on each corner as shown in Figure 11.

Each 8x8 ft. plywood wall was constructed with plywood on both sides except for Test 7 which used plywood on one side. Typical 2x6 in. studs and 2x6 in. blocking was used. For the exterior side connecting to steel frame 3x6 in. redwood plate was used and for interior side double 2x6 in. plate was used. See Figure 12 for typical plywood framing. All plywood was exterior grade and the framing material was No. 1 construction grade. The studs were nailed to redwood plate with 2-10d toenails. The studs were nailed to double plate with 2-20d toenails. The double plate was nailed with 16d toenails at 9 in. c-c. The plywood was nailed in the perimeter with 8d at 4 in. c-c for Test 1, 2, 3, and 7, 8d at 6 in. c-c for Test 4, 10d at 6 in. c-c for Tests 5 and 6, 6d at 4 in. c-c for Panel 8. The plywood was nailed to redwood with 8d at 4 in. c-c for Panels 1, 2, 3, 4, and 7, 10d at 4 in. c-c for Panel 6, and 6d at 3 in. c-c for Panel 8. The plywood was nailed to double 2x6 in. plate with 8d at 4 in. c-c for Panels 1, 2, 3, and 7, 8d at 6 in. c-c for Panel 4, 10d at 6 in. c-c for Panels 5 and 6, and 6d at 4 in. c-c for Panel 8. The plywood thickness was 3/4 in. both sides for Test 1, 3/8 in. both sides for Tests 2, 3, 4 and 8, 1/2 in. both sides for Tests 5 and 6, and 3/8 in. one side only for Test 7.

Young and Medearis used a conversion, by means of symmetry, from single-span action to that of two cantilevers whose origins are at the single span center line; and rearranging of the cantilevers to give the final test interpretation, that of two cantilever shear walls, side by side, and subjected to a total shear load, P_c. (Figure 13).

The testing machine frame, was equipped with two special channel end supports designed to take reactions from reversed loading. Cyclic loading was applied to the double shear wall, through a vertical channel on the axis of symmetry, by hydraulic

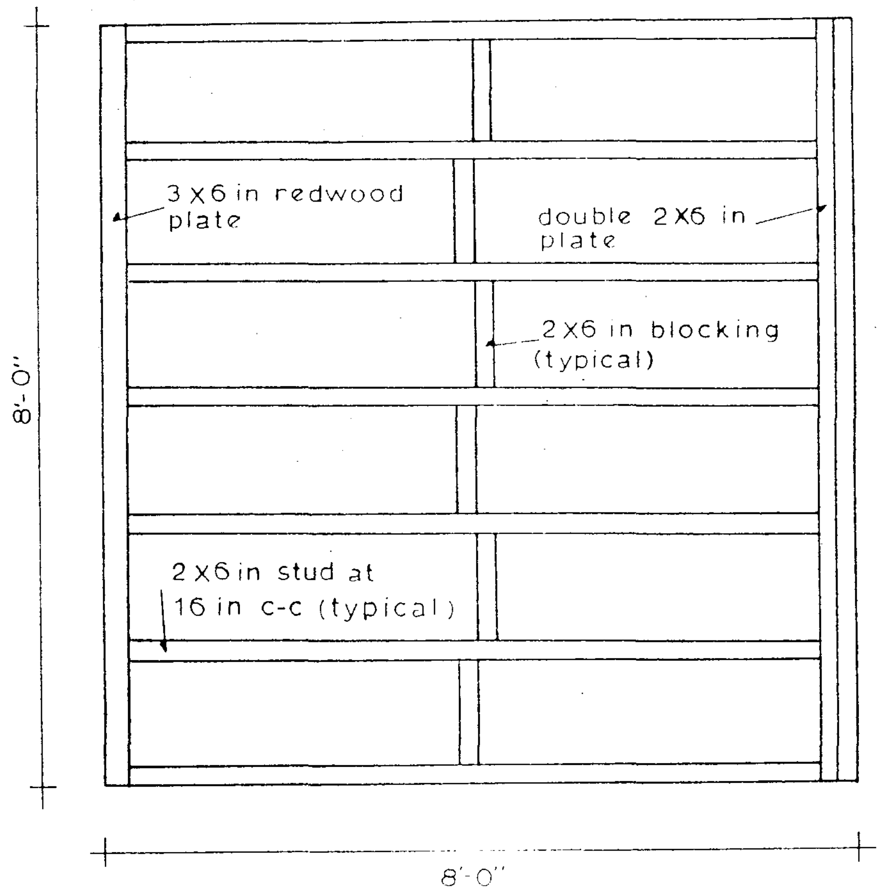

Figure 11. Typical plywood framing [518]

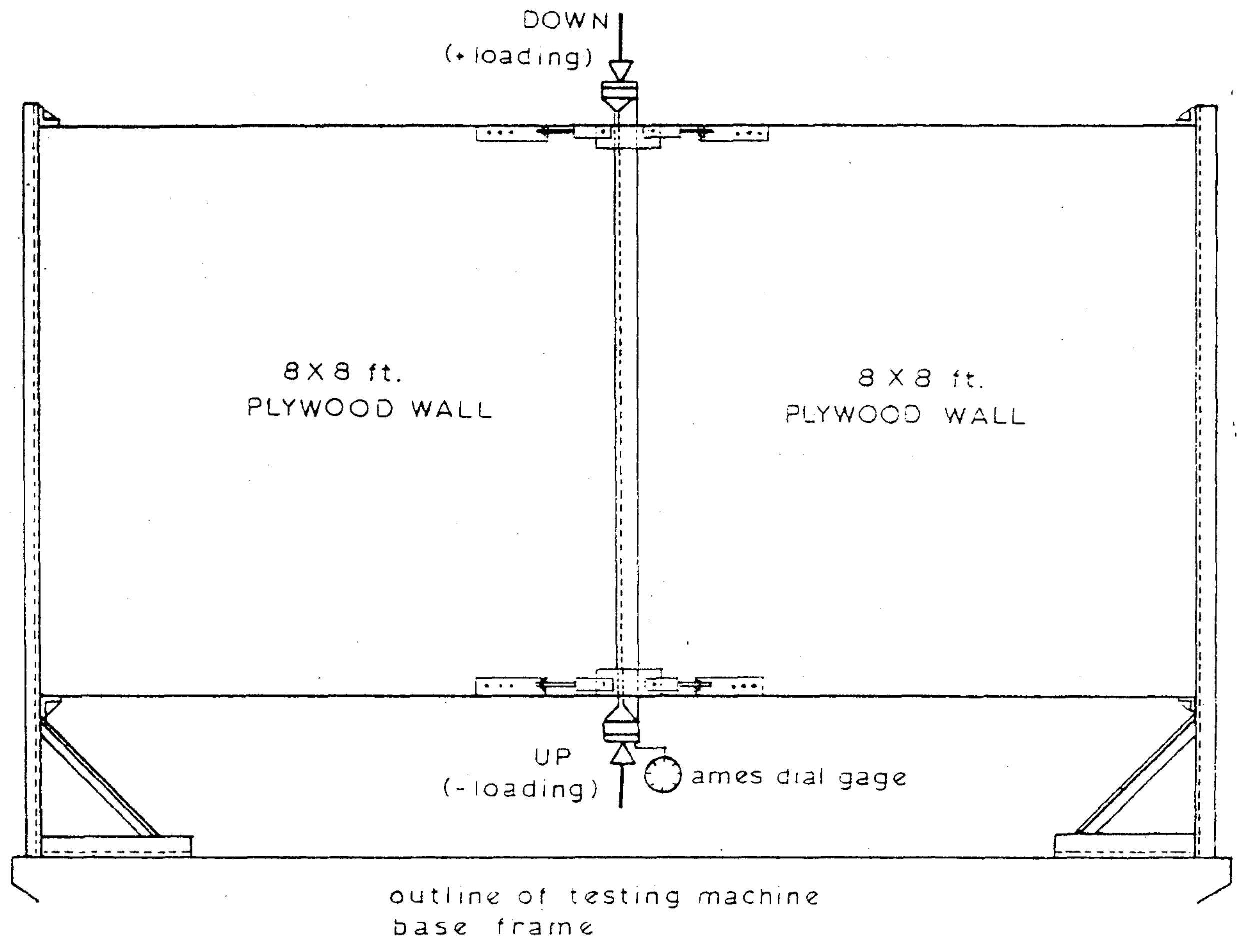

Figure 12. Panel test showing location of deflection gauges [518]

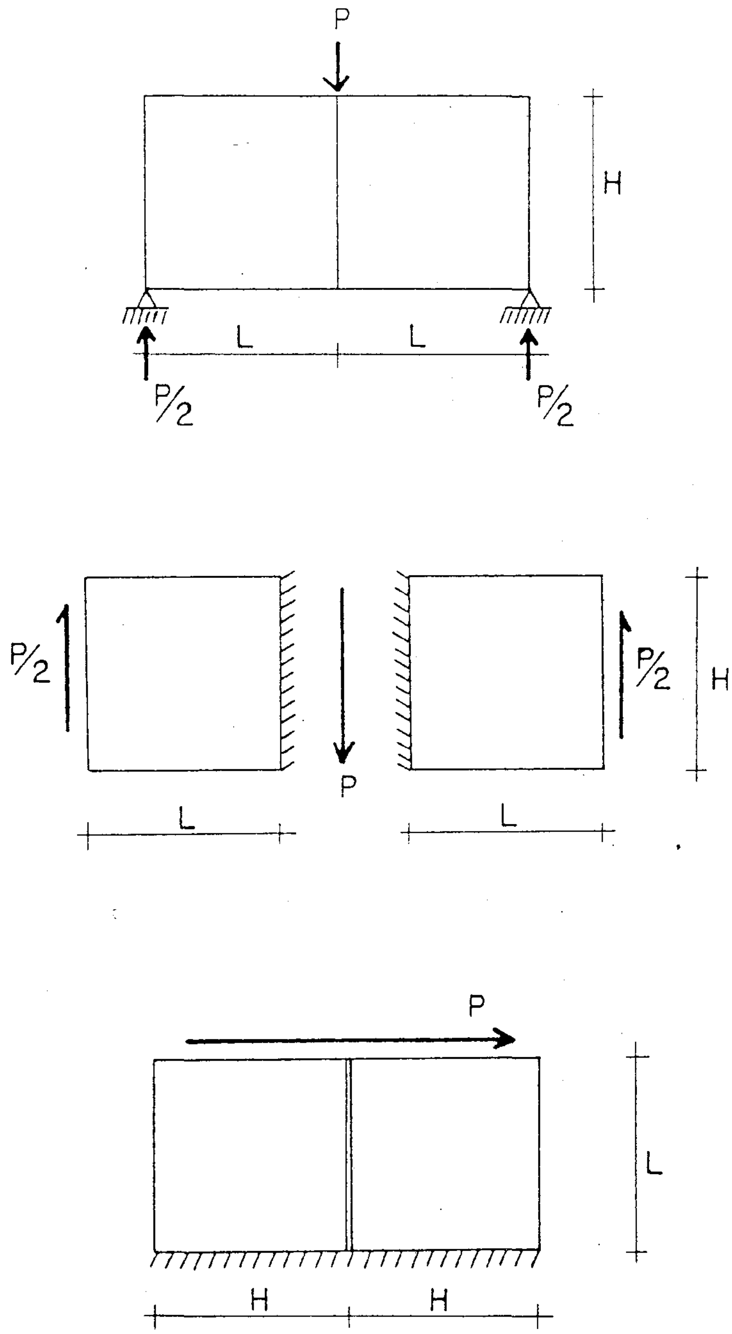

Figure 13. Test interpretation [518]

jacks. Fifty-kip capacity SR-4 load cells were used in conjunction with the jacks. The experiments were conducted by using peak load increases, from one cycle to another, of 4 kips. The load was applied in increments of 1/4 the peak, or maximum, load to be attained per given cycle and unloaded using 1/2 the maximum load.

The true test panel center deflection was determined by using three Ames dial gages, one at each end support, and one at the center. The support gages were utilized to account for movements of the deflection reference line resulting from end-support crushing of the wood. The true center deflection, Δ_c, was obtained from the following equation:

$$\Delta_c = \Delta'_c - (\Delta_L + \Delta_R) / 2 \quad (3\text{-}5)$$

where

Δ_c = true center deflection
Δ'_c = the center dial gage reading
Δ_L = the left support dial gage reading
Δ_R = the right support dial gage reading

The larger deflections of the panels, resulting from above-yield loading, always exceeded the capacity of the center dial gage. Thus, to measure these high-load deflections, a transit was used in conjunction with a 50 divisions/inch scale which was mounted on the center loading channel.

Test 3 (identical to Panel 2) was planned to investigate panel deterioration effect, as a result of being subjected to cyclic loading at the design load prior to subsequent cyclic loading to ultimate. Test 6 was planned to investigate the possibility of re-nailing a panel that had already been subjected to a near ultimate load. This was accomplished by taking the previously tested #5 panel, jacking it back to its neutral configuration, and then re-nailing it.

3.5 Research performed by Porter and Easterling [378]

The following discussion of data formulation on SDRC diaphragms was taken from Reference [378]. Experimental data result of this research is included in the Appendix A, Section 8.3.1, Table A8 of this report.

A series of 32 full-scale SDRC diaphragms were tested at ISU as part of a two phase research. The first phase included Tests 1-9 and was performed by Porter and Greimann [390]. The second phase included Tests 10-32 and was performed by Porter and Easterling [378]. Variables included deck type, deck thickness, fastener type, number of fasteners, concrete thickness, depth to span ratio, load combinations of in-plane and gravity loads and edge member size. Each of these parameters were investigated with regard to their influence on behavior and strength of SDRC diaphragms.

Numerous SDRC diaphragm parameters were varied and tested on the experimental sections of the study. Diaphragms 1-21 were 15 ft. x 15 ft. in plan while Diaphragms 22-32 were 15 ft. x 12 ft. in plan. Diaphragms 1-31 were constructed with W24x76 steel sections and Diaphragm 32 was constructed with W14x22 steel sections. All diaphragms except diaphragm 26 were constructed with normal weight concrete, with 26 being constructed with structural lightweight concrete. Deck sections were classified as different if the profile, deck thickness or embossment configuration is unlike any other.

A horizontal cantilever test frame was designed in the first phase of the research [30] and is shown schematically in Figure 14. The north to south dimension of the span of the diaphragm was the 12 ft. dimension. The south edge of the test frame was constructed using three reinforced concrete blocks. These blocks served as the reaction edge and were anchored to the structural test floor by post-tensioning 2 in. diameter high strength steel rods. Connection of the framing members to each other and to the south edge was made using flexible tee sections.

Load was applied via two reversible hydraulic actuators as shown in Figure 14. Each actuator has a capacity of approximately 200 kips and both are driven with a closed loop servo-valve controlled system. Six diaphragms were loaded with both in-plane and vertical load [335]. Vertical load was applied to 20 distributed neoprene pads on the surface of the specimen. The amount of vertical load applied was chosen to model an equivalent distributed load based on equivalent shear area in the one-way direction (parallel to the corrugations).

Data were collected for each of the diaphragms utilizing mechanical dial gages, electrical resistance strain gages, electronic displacement transducers (DCDT), and load cells. The electrically recorded data were collected using a 150 channel data acquisition system (DAS). Components of the system are micro-computer, digital plotter and printer and a 150-channel digital voltmeter with five independent power supplies. Figure 15 shows a schematic of the experimental testing arrangement.

Reversed cyclic loading was used for all test specimens in the second phase of the research. The tests were displacement controlled using the DCDT in the northeast corner in line with the push beam as the feedback to the closed loop system. At each level of displacement in the displacement history a minimum of three complete cycles was made. The criteria for increasing to the next level was that the load had to stabilize within a certain margin. This margin was defined as being less than a five percent change in load from the previous cycle at the same displacement. An initial displacement level of 0.025 in. was selected for the experimental

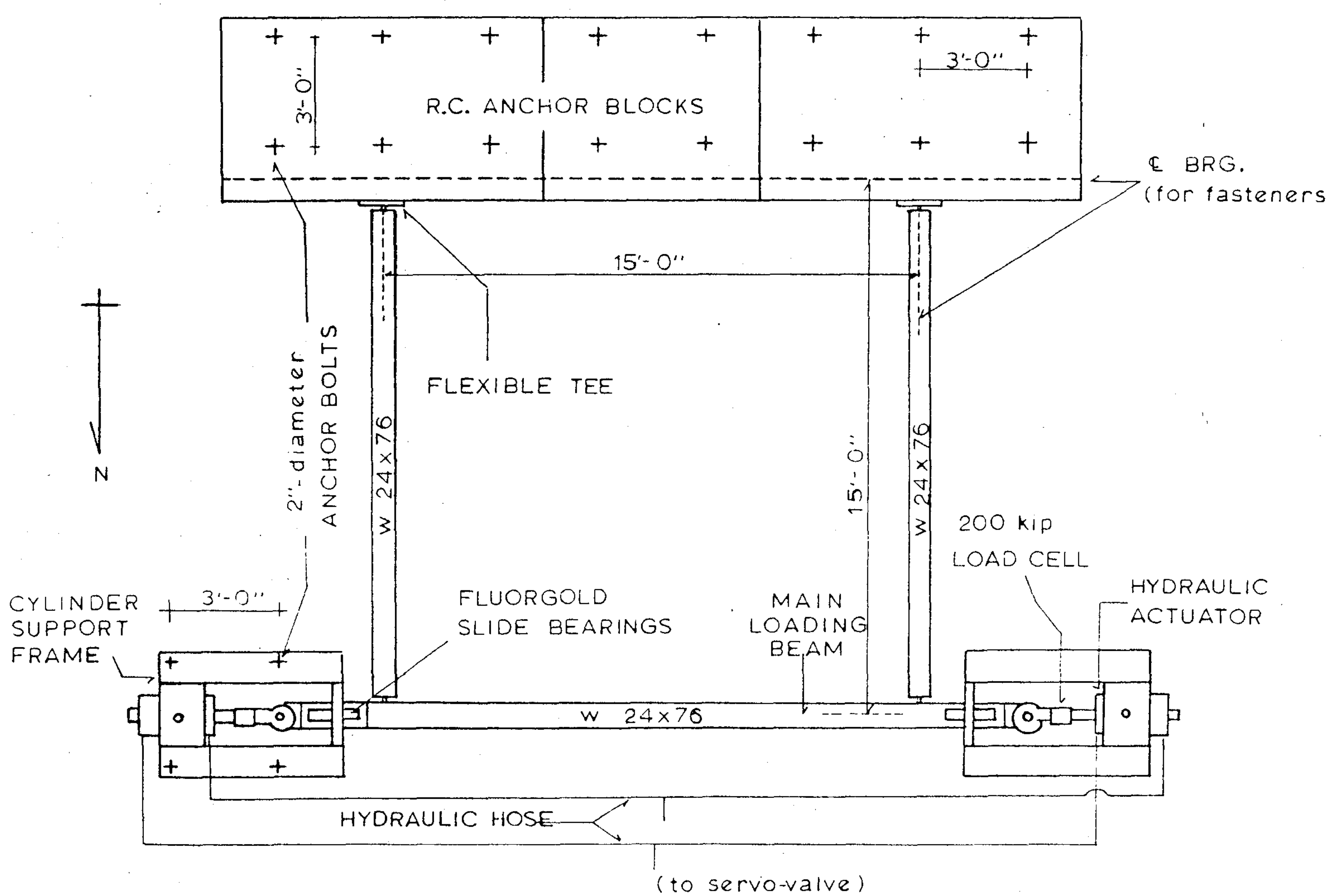

Figure 14. Diaphragm test frame schematic [378]

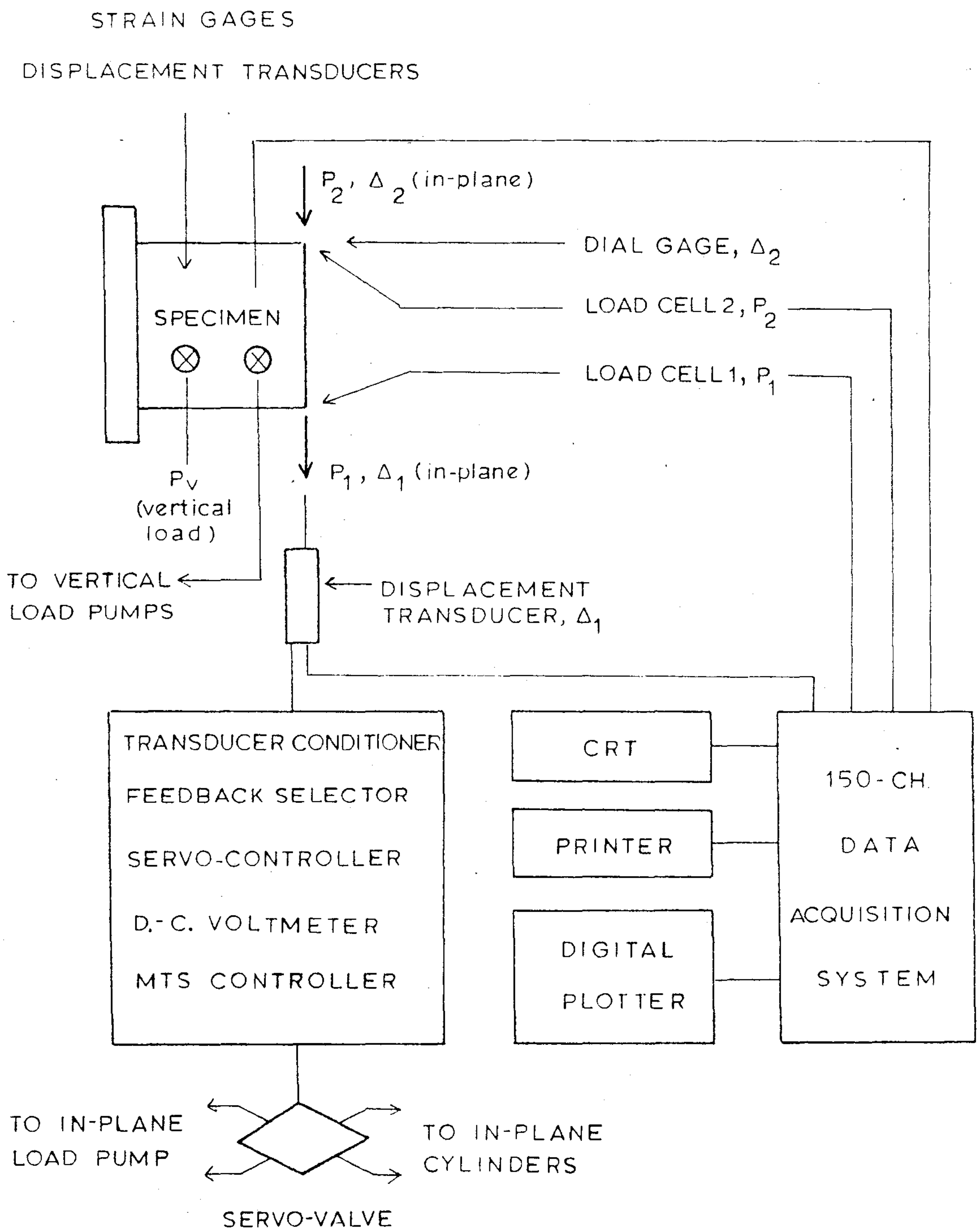

Figure 15. Schematic of servo-hydraulic testing system [378]

program. This displacement was thought to be within the elastic range of behavior for the SDRC diaphragms.

Initial or elastic stiffness values for each diaphragm were determined. These values were determined based on the first nominal displacement to 0.025 in. The total load for both actuators was divided by the displacement of the controlling DCDT in order to obtain the initial stiffness. No adjustments to the experimental stiffness values for test frame stiffness were made, since pre-test frame calibrations without an attached diaphragm, showed the frame to be relatively flexible.

Load and displacement were continuously monitored at intervals of approximately one second during the displacement histories. This enabled the determination of peak load and corresponding displacement between load points. No adjustment to strength values based on test frame strength, without an attached diaphragm, were made, since frame calibration indicated that the load carried by the base frame was approximately 1 kip. This contribution was deemed negligible.

3.6 Research performed by ABK, a Joint Venture [2,3]

The following discussion of data formulation on wood, steel deck, and SDRC diaphragms was taken from Reference [2]. Experimental data result of this research is included in the Appendix A, Section 8.1.3, Table A4, and Section 8.2.2 Table A7 of this report.

A series of 14 configurations of 20 ft. by 60 ft. diaphragm specimens were tested by ABK A Joint Venture. The testing was conducted at the El Segundo Structures Laboratory of the North American Aircraft Division of Rockwell International Corporation. From 14 tests performed, 11 were on wood diaphragms, two were steel deck diaphragm, and one was steel deck concrete filled diaphragm.

Four 20 ft. x 60 ft. frames were constructed to accommodate the seven prime wood sheathed diaphragm configurations. These had 4" x 12" edge and end members with 2" x 12" joists at 24 inches on center as shown in Figure 16.

All framing lumber was 2" x 12" and 4" x 12" Douglas Fir No. 1, and the blocking was 2" x 4" Douglas Fir No. 2. The lumber sheathing was 1" x 6" Douglas Fir, Construction Grade. The straight or diagonal sheathing boards were laid so that there were not less than two boards between joints on the same bearing and not less than two bearings between joints in adjoining boards. These were joined with three 8d nails at ends of each board and two 8d nails at all other bearings. The plywood sheathing conformed to U.S. Product Standard PS-1-74, Structural I, Exterior Glue, Grade C-D, Douglas Fir. Common wire nails were used for the most part,

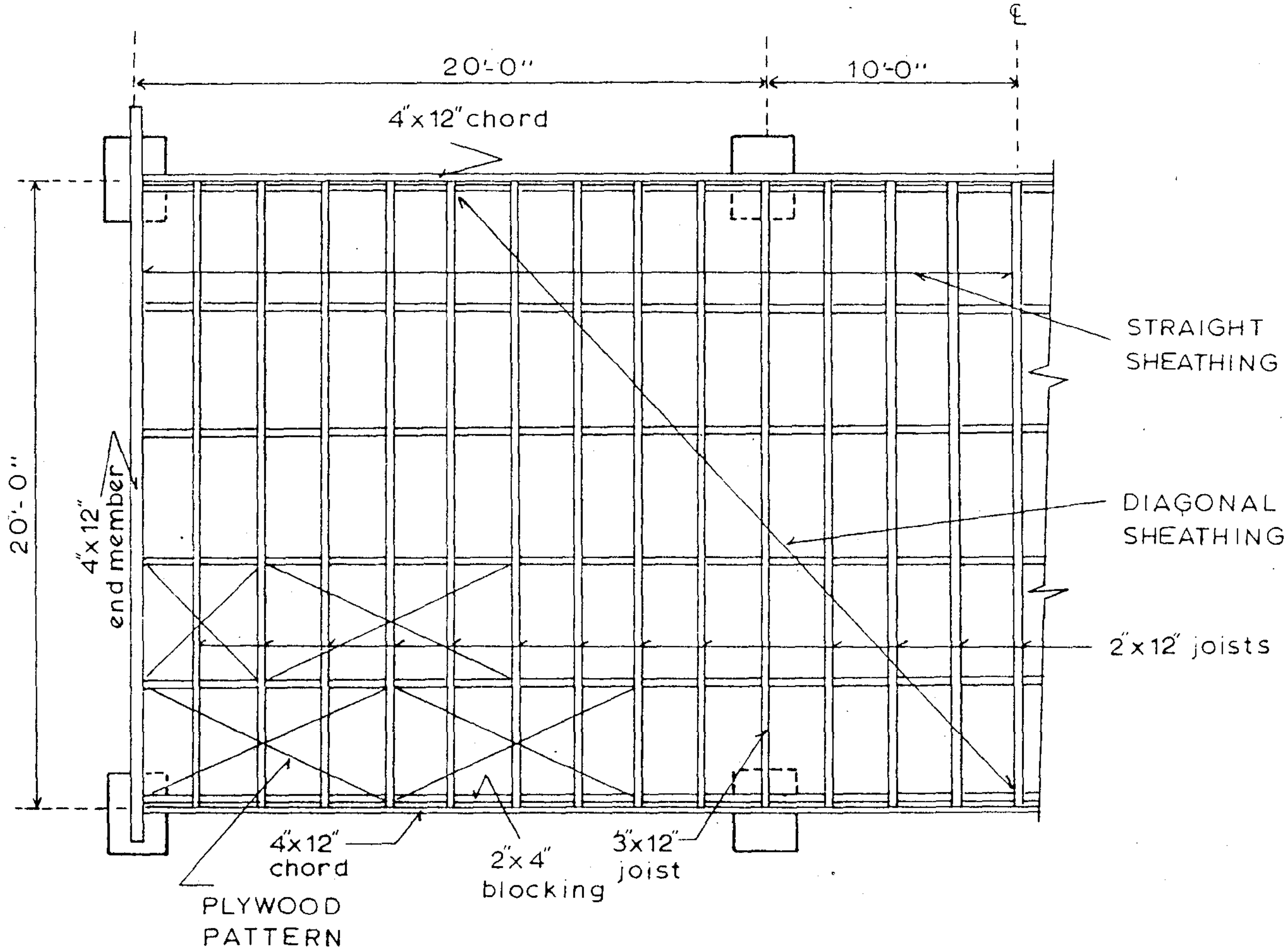

Figure 16. Framing for wood sheathed diaphragms [2]

and duplex head nails were used where they did not interfere with overlaying material. Power driven "short" plywood nails had the same gage as for specified nail.

Diaphragm C was constructed of 1/2 inch thick plywood, unblocked, with built-up roofing applied. The plywood sheets were laid across the joists and staggered. The plywood was nailed with 8d nails at 6 inches on center at the edges of the sheets and at 12 inches on center for intermediate bearings. 8d nails at 6 inches on center were used to connect the edges of the plywood sheets to the 2" x 4" blocking along the 4" x 12" chord members to provide an unchorded condition.

Diaphragm D was a rework of Diaphragm C. After Diaphragm C was tested, about 12 ft. of the roofing at each end was rolled back to observe the plywood sheathing. Any nail torn out of the edge of the plywood was replaced by another nail into adjacent solid plywood. The roofing was laid back in place and renailed with roofing nails through the roofing 6 inches o.c. along the end 20 feet of the two edges and across the 20 ft. ends, and field nailed at 18 inches o.c. each way for 20 ft. at each end. In addition, the edges of the plywood were nailed to the 4" x 12" chords with 6d nails at 6 inches o.c. through the roofing.

Diaphragm B was a rework of Diaphragm D after it was tested. Five full sheets and five half sheets at the two ends of the diaphragm were replaced and nailed. In addition, the other plywood sheets within 20 feet of the ends of the diaphragm were renailed. The chord nailing was also replaced with 8d nails at 6 inches o.c.

Diaphragm E was constructed of 1" x 6" straight lumber sheathing with built-up roofing. The sheathing was laid across the joists in the long dimension of the diaphragm. The edge board was nailed to the 2" x 4" blocking rather than the 4" x 12" chord to simulate an unchorded diaphragm. Prior to testing, the roofing was nailed with roofing nails at 6 inches o.c. to the edge and end 4" x 12".

Diaphragm E1 was a retest of Diaphragm E by renailing the roofing for 20 ft. at each end with roofing nails through 2-3/4 inch diameter by 30-gage metal washers. Renailing was at 6 inches o.c. along the edges and ends and at 18 inches o.c. each way in the interior field.

Diaphragm H was a rework of Diaphragm E1 after it was tested. The entire diaphragm was overlain with 5/16 inch thick plywood. The plywood was nailed through the sheathing with 8d nails at 6 inches o.c. at the ends of the sheets and at 12 inches o.c. at the intermediate bearings. The exterior edges of the plywood were nailed to the 4" x 12" chord with 8d nails at 6 inches o.c. to provide a chorded condition.

Diaphragm I was constructed of 1" x 6" lumber sheathing laid at 45 degrees diagonally to the diaphragm and covered with built-up roofing. The boards were nailed to the 2" x 4" blocking rather than the 4" x 12" chords to simulate an unchorded diaphragm. Prior to testing, the roofing was nailed with roofing nails at 6 inches o.c. to the edge blocking and the end 4" x 12".

Diaphragm I1 was a retest of Diaphragm I by renailing the roof for 20 ft. at each end with roofing nails through 2-3/4 inch diameter by 30-gage metal washers. Renailing was at 6 inches o.c. along the edges and ends, and at 18 inches o.c. each way in the interior field.

Diaphragm K was a rework of Diaphragm I1 after it was tested. The broken boards were replaced and all broken and bent nails were renailed. 1" x 6" straight lumber sheathing was laid across the joists in the long dimension of the diaphragm. The straight sheathing was nailed through the diagonal sheathing with the specified nailing.

Diaphragm N was constructed of 1/2 inch thick plywood. Flat 2" x 4" blocking was provided between the joists along the edges of the plywood sheets. The plywood was nailed at all edges with 8d nails at 4 inches o.c. and at 12 inches o.c. to the intermediate bearings. The nailings to the 4" x 12" edge members provided a chorded diaphragm.

Diaphragm P was constructed on the framework of Diaphragm N after it was tested. The 1/2 inch plywood was removed and two layers of 3/4 inch plywood were nailed to the frame. The lower sheets were laid across the joists as in the previous case of the 1/2 inch plywood diaphragm. These sheets were nailed at all edges with 8d nails at 4 inches o.c. and at 12 inches o.c. to the intermediate bearings. The upper sheets were laid at right angles to the lower sheets and nailed at all edges with 8d nails at 4 inches o.c. and at 12 inches o.c. in rows 2 feet apart. The layout of the sheets was slightly offset from that in Diaphragm N to allow end nailing into areas of the 2" x 12" joists that had no previous nailing.

A separate 20 ft. x 60 ft. structural steel frame was provided for each of the two prime steel deck diaphragm configurations. These had W12x27 edge, end, and interior beam members and W12x19 joist members as shown in Figure 17. This provided a three-span condition for the steel decks.

The structural steel conformed to ASTM A36, and the bolts conformed to ASTM A307. ASC Pacific, Inc. - Type B (interlock), 20 gage, galvanized, was used for the unfilled deck diaphragm and ASC Pacific, Inc.-type B(Hiform), 20 gage, galvanized,was used for the diaphragm with concrete fill. The connections at the support members were 1/2 inch diameter effective plug welds at each flute. For the marginal and parallel members, 1/2 inch diameter effective

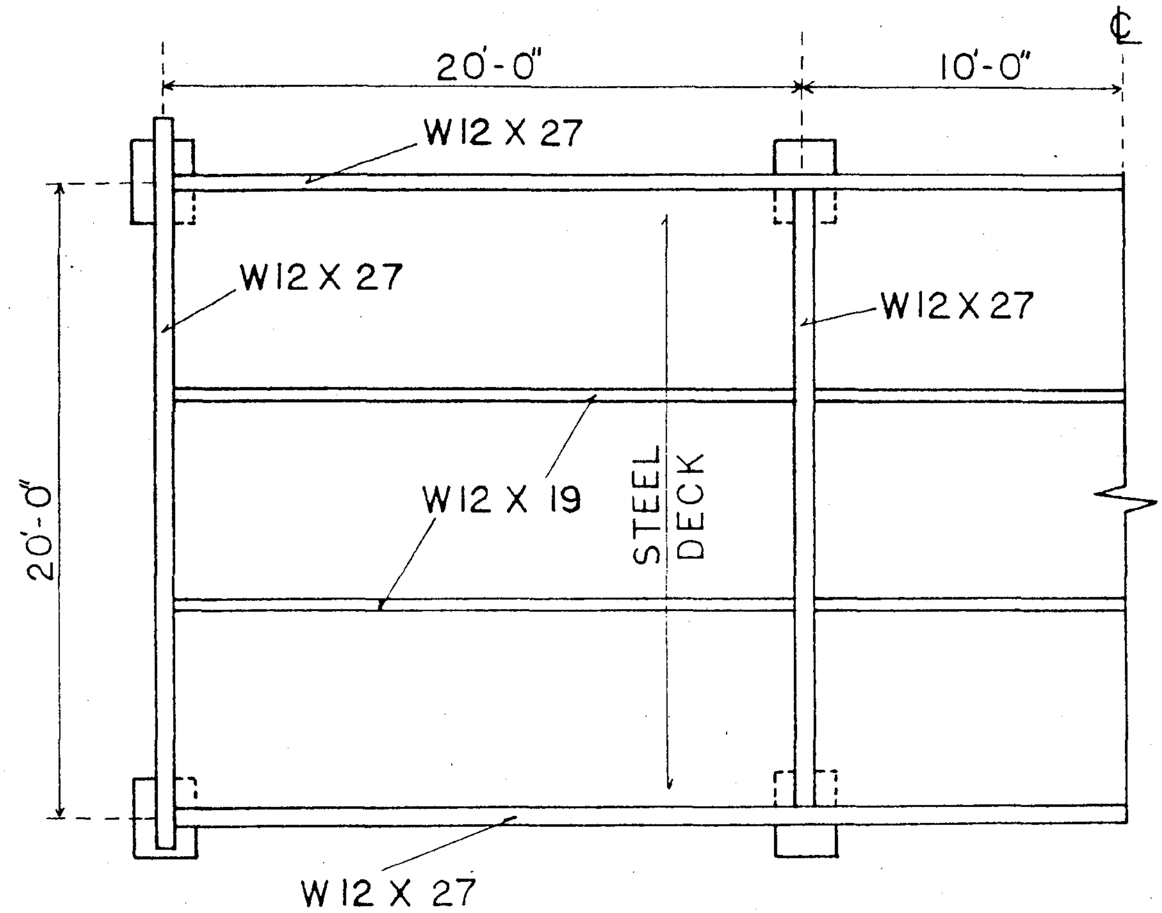

Figure 17. Steel diaphragm testing [2]

plug welds at 12 inches on center were used. The side laps were button-punched at 18 inches on center.

The concrete fill was standard weight concrete with an ultimate compressive strength f'_c of 3000 psi at 28 days. The concrete was reinforced with 6" x 6"/W1.4xW1.4 electric welded mesh. The concrete was covered with an impervious membrane immediately after placing.

Diaphragm Q was constructed of twenty-four 30-inch wide by 1-1/2 inch deep, 20-gage steel deck panels laid over the structural steel framework. The deck was attached to the steel frame members with the specified plug welds and the seams were button punched 18 in. on center. The splice bars were not attached to simulate an unchorded condition.

Diaphragm R was a rework of Diaphragm Q after it was tested. Broken plug welds were replaced or equivalent fillet welds were placed on the end of the deck panels at the edge (chord) members of the framework. The seams were button punched at 6 inches on center for the entire diaphragm. The splice bars were bolted in place to provide a chorded condition.

Diaphragm S was constructed of twenty-four 30-inch wide by 1-1/2 inch deep, 20-gage steel deck panels laid over the structural framework. These panels had embossments along the webs of the flutes to provide composite action with the concrete fill. The deck was attached to the steel frame members with the specified plug welding and the seams were button punched as specified. The splice bars were bolted in place to provide a chorded condition. The specified electric welded wire mesh was laid on the deck and 2-1/2 inches of concrete was placed over the top of the flutes. The concrete was screeded and bull floated to grade and covered with a black polyethylene film to retain the moisture for curing.

The test setup for the quasi-static, in-plane displacement of the diaphragms is shown in Figure 18. The diaphragm specimens were supported on eight low friction roller assemblies and were displaced, in-plane, at the two ends by programmable hydraulic actuators. For the quasi-static tests, removable reaction pillars were moved into place at the diaphragm's third points and the diaphragm was cyclically displaced in its plane by the hydraulic actuators.

The basic instrumentation for the measurement of the diaphragm responses and forcing functions is shown in Figure 19 and consisted of displacement sensors, accelerometers and load cells; and all of the instrumentation measured in-plane responses and forcing functions. The displacements were measured using string potentiometers. The displacement sensors, including feedback deflection sensors, were mounted to a stable reference frame that was independent of the frame for the forcing system. This type of

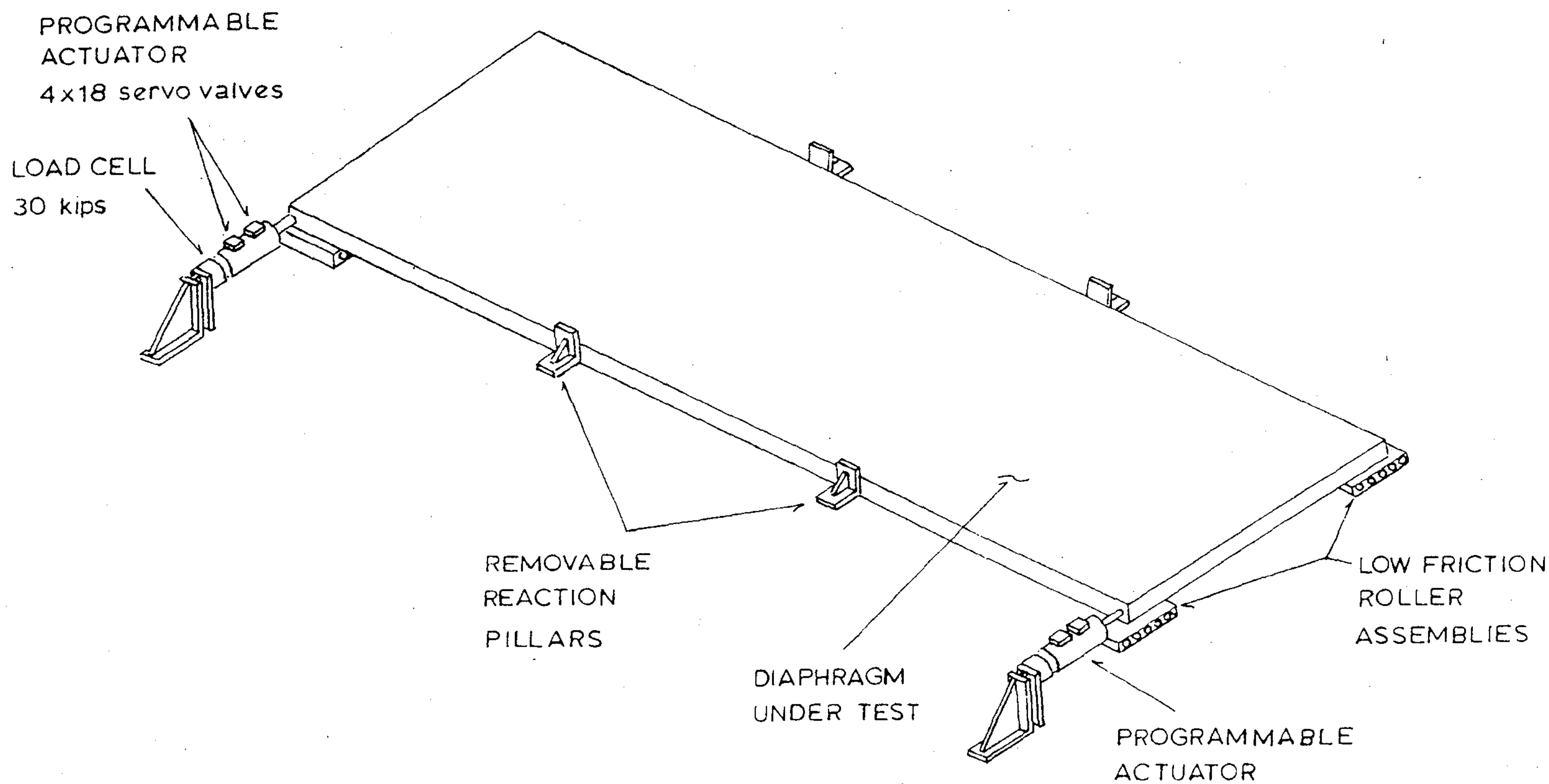

Figure 18. Schematic of the test setup for quasi-static testing of diaphragms [2]

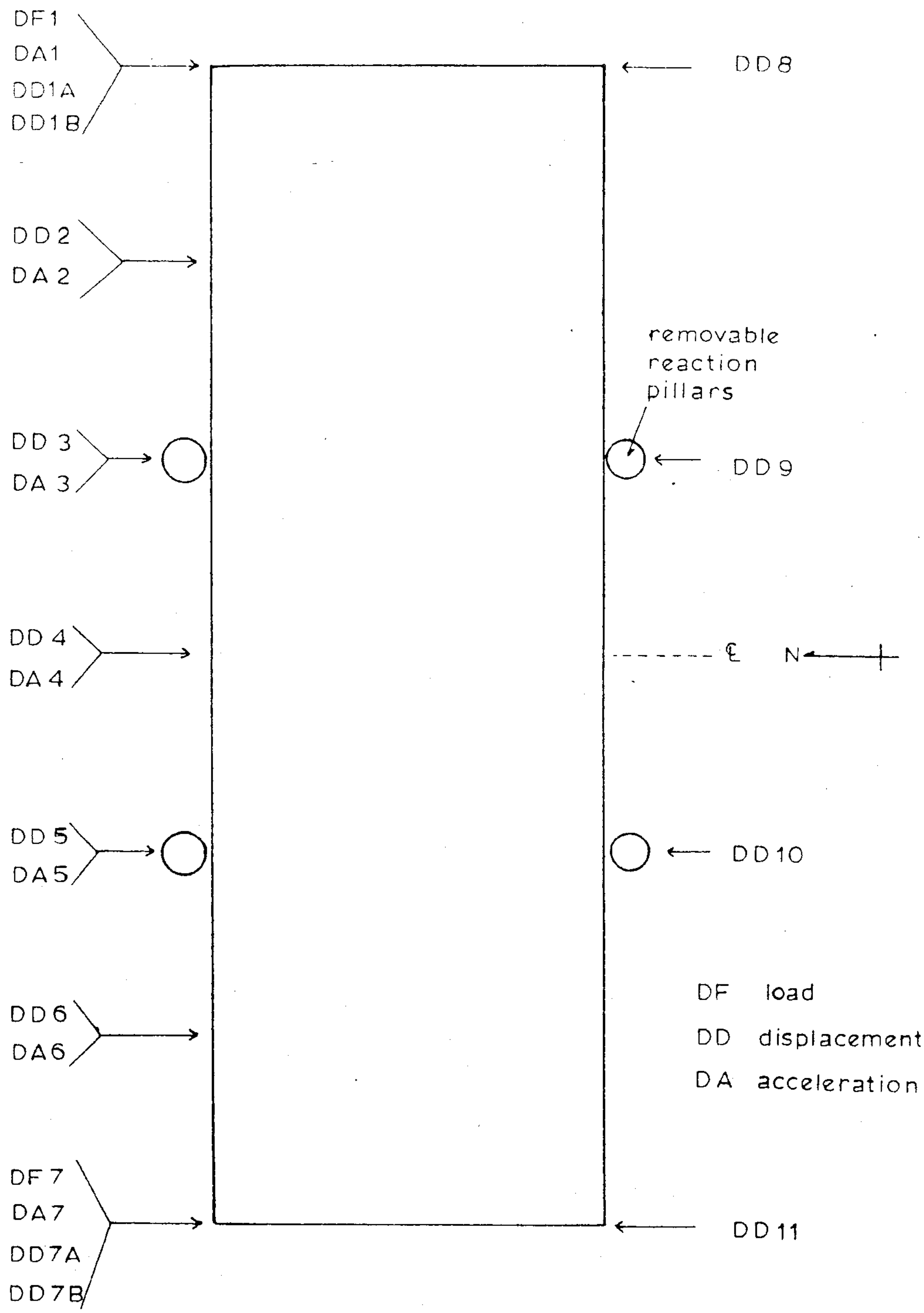

Figure 19. Diaphragm instrumentation [2]

instrument mounting was used to eliminate the need to account for the flexibility of the actuator reaction stands. The data from each instrument was recorded on magnetic tape in digital form.

The quasi-static loading cycles were normalized in terms of multiplicative factors of the basic static deflection delta of each diaphragm. Diaphragms E and E1 had deltas of 0.16 inches. Diaphragms H, C, D, B, N, Q, and R, had deltas of 0.4 inches. Diaphragms I and I1, had deltas of 0.72 inches. Diaphragm K had delta of 0.3 inches. Diaphragm P had delta of 0.25 inches, and Diaphragm S had delta of 0.1 inches. The delta for each diaphragm corresponds to an estimate of the approximate elastic limit of the diaphragm.

The test results reported by ABK [2] includes measured data from instrumentation. Other reports presented by ABK [3] includes the interpretation of diaphragm tests and give us initial stiffness, strength and pinch force for 12 of 14 diaphragms tested.

3.7 Research performed by Porter, et. al. [376]

Experimental data result of this research is included in the Appendix A, Section 8.4.1, Table A9 of this report.

Sixteen hollow-core diaphragm tests have been conducted at the Structures Laboratory at Iowa State University. The configuration of the test slabs is shown in Figure 20. Each of these tests employed connecting the planks to a cantilever steel I-beam loading frame with Nelson studs and high strength grout. A list of the parameters involved with each test is found in Table 1.

All slabs were subjected to the sequential phased displacement program as presented in Figure 21. Complete hysteretic curves were recorded for perimeter displacements and seam slips at several locations. One of these hysteretic curves is shown as an example in Figure 22. An envelope of the entire hysteretic behavior was established for both the initial (virgin) and the stabilized curves, as shown in Figure 23.

Comparisons were made between the significant limit state strengths associated with the first major event (FME) and other key events (or modes of failure), stiffness, displacements, and peak strengths.

The plank orientation was determined to be one of the most important parameters affecting the strength of the untopped diaphragms. The perpendicular orientation provide for higher FME load, and peak strength. For the topped slabs the orientation effects were dependent on the number of sides connected to the loading frame. The tests with two sides connected produced greater FME and peak load, while the tests with four sides connected did

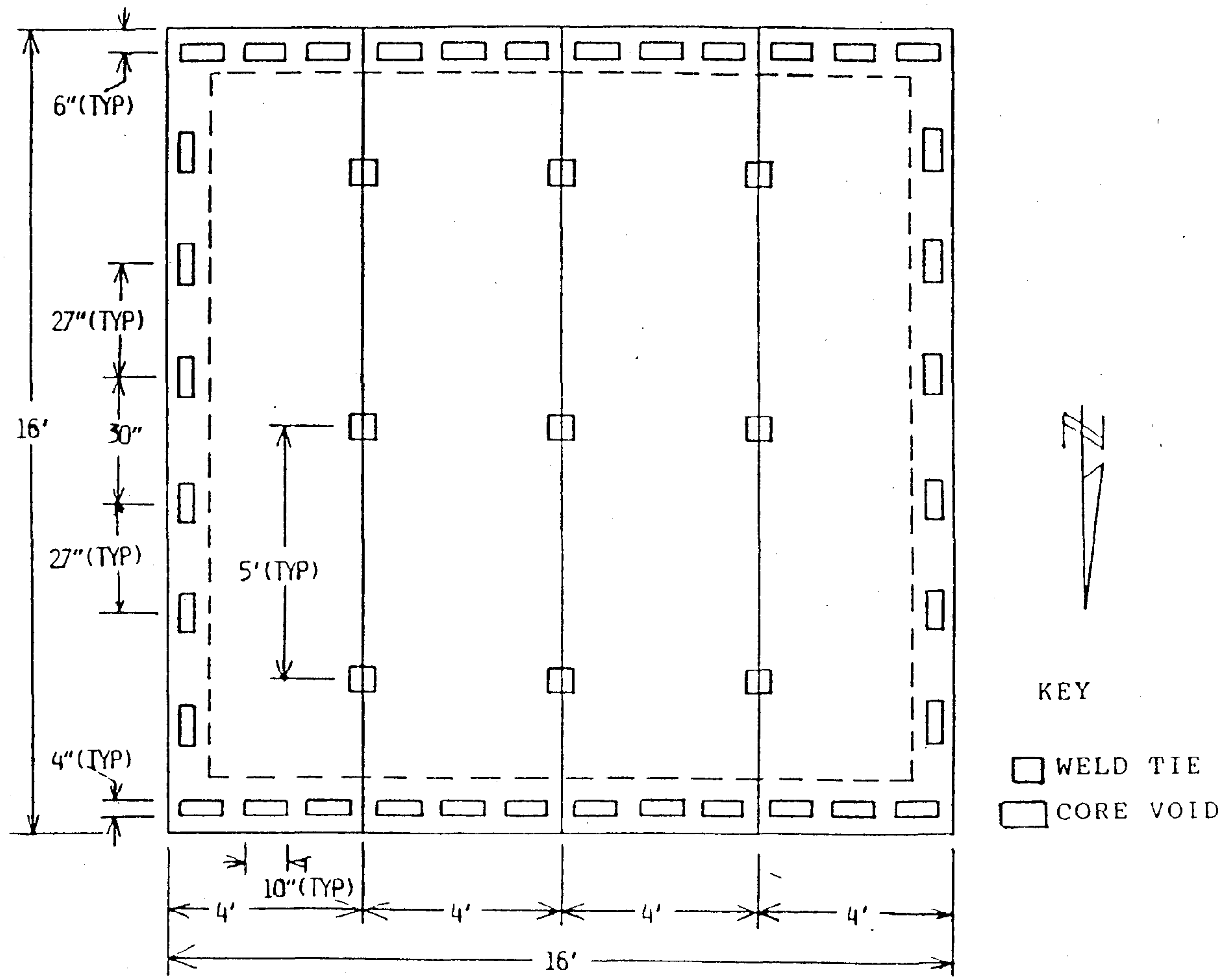

Figure 20. Typical dimensions for all diaphragms (except Test #3) [481]

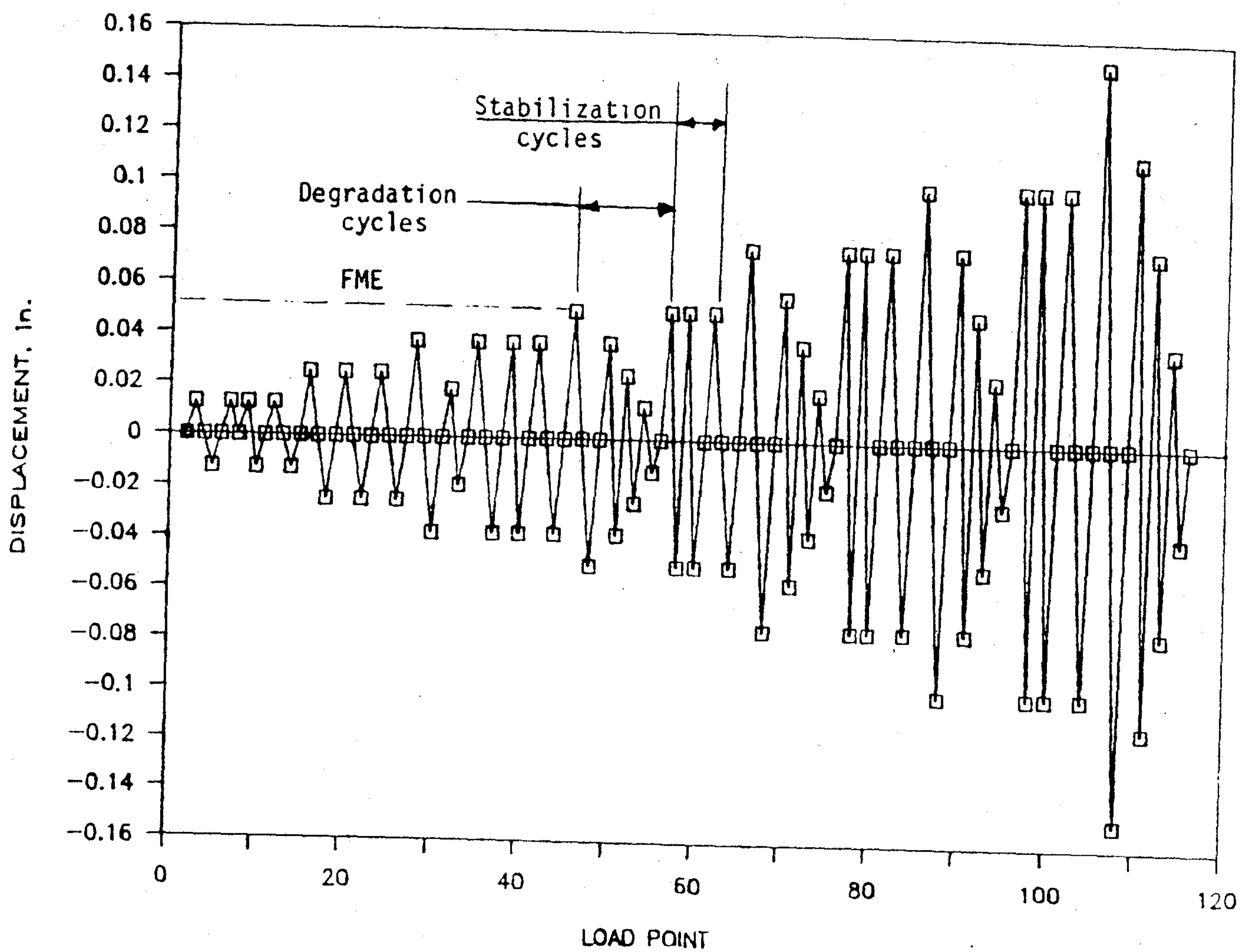

Figure 21. Example of SPD load program [481]

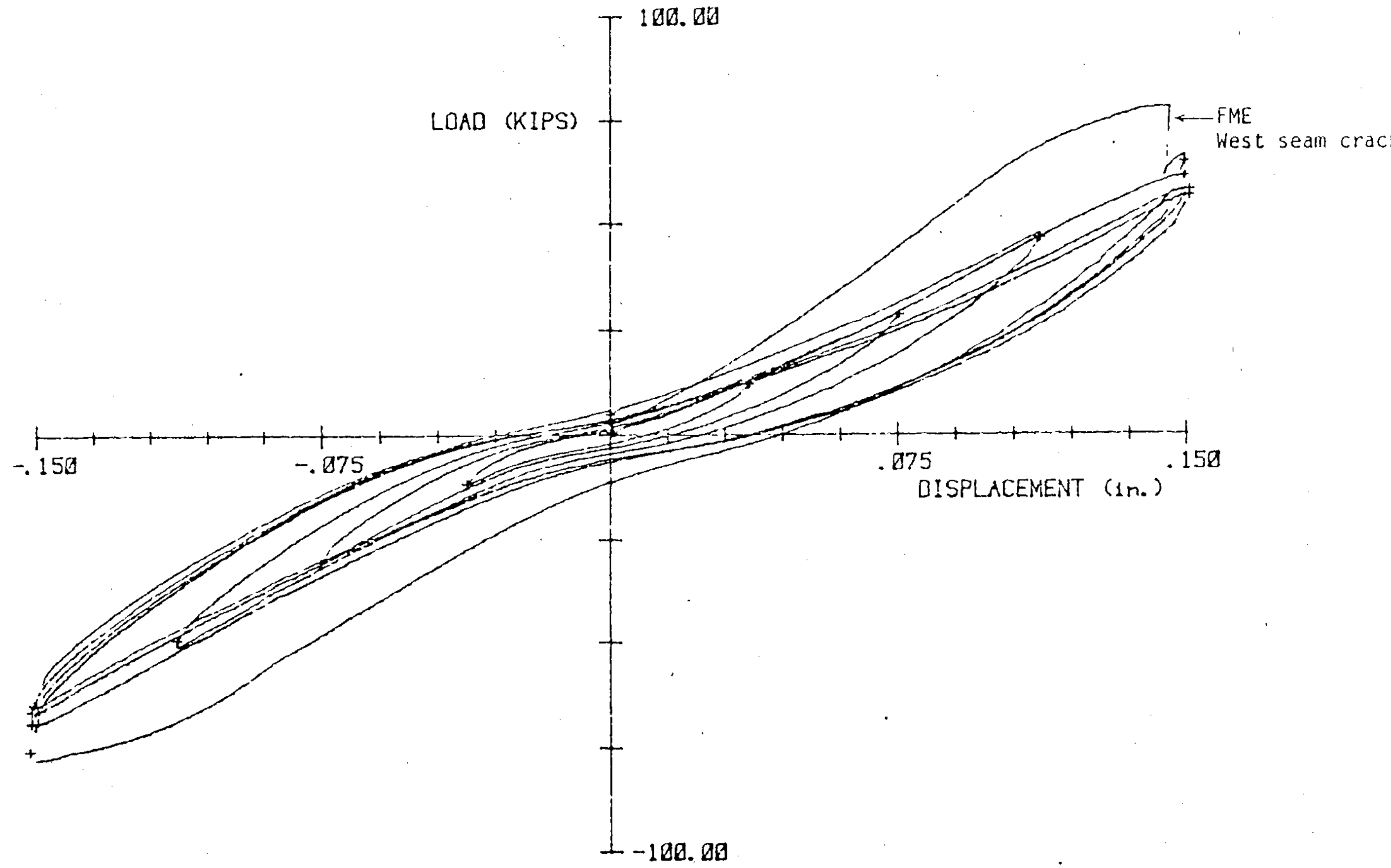

Figure 22. Example of hysteretic curves [481]

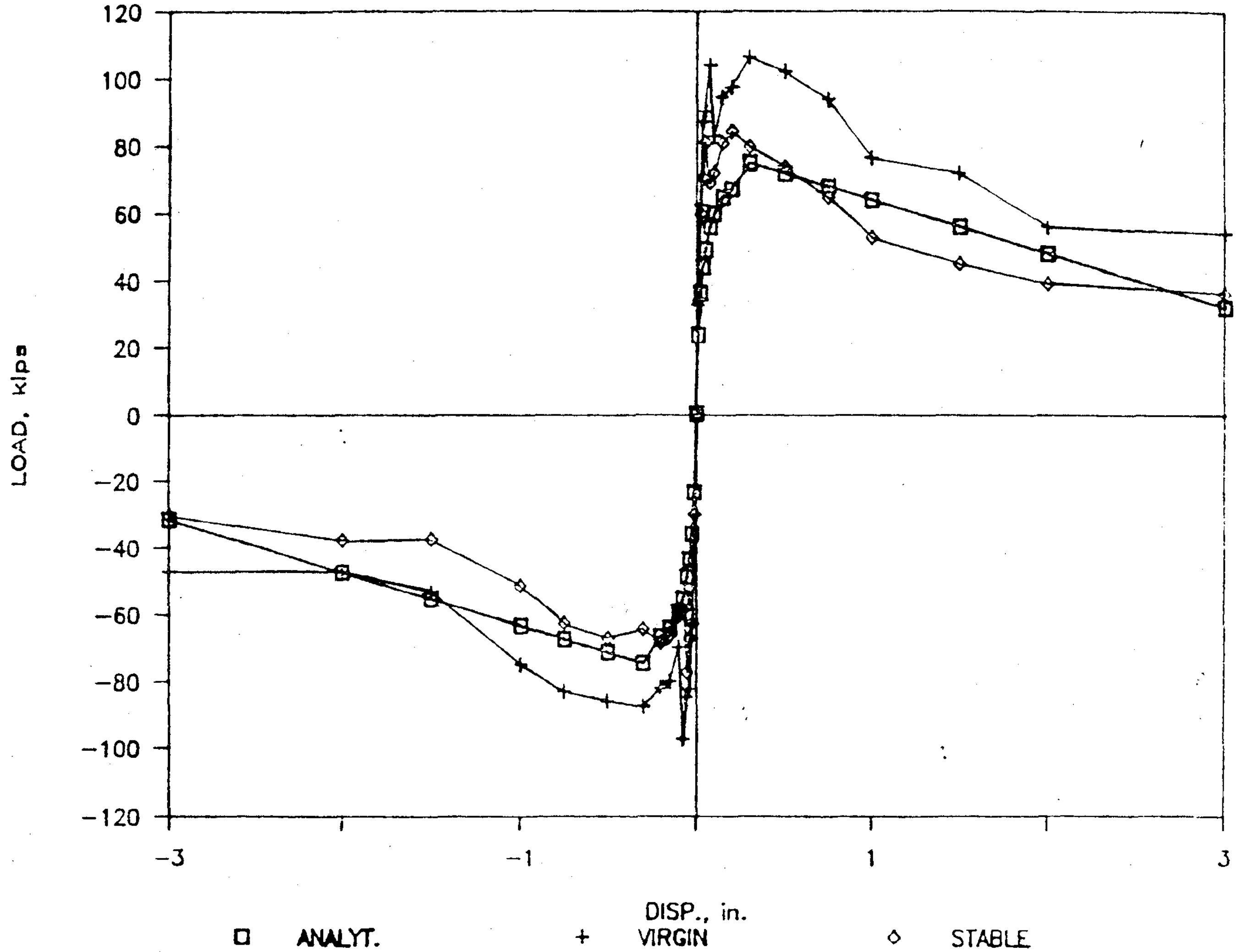

Figure 23. Example of envelope curves [481]

Table 1. **SUMMARY OF PARAMETERS FOR THE DIAPHRAGM TESTS** [376]

Test No	Sides Connected	Orientation	Topping	Plank Depth (in.)
1	2	EW	NO	8
2	2	EW	NO	8
3	2	NS	NO	8
4	2	NS	NO	8
5	4	NS	NO	8
6	4	EW	NO	8
7	3	EW	NO	8
8	2	EW	NO	8
8B	2	EW	YES	8
9	2	NS	NO	6
10	4	EW	NO	6
11	2	NS	NO	12
12	2	NS	YES	8
13	4	NS	YES	8
14	4	EW	YES	8
15	4	NS	NO	8

Note

1) Test #15 had 15 weld ties per seam.
2) Topped tests did not have any weld ties.
3) Untopped tests (with the exception of #15) had 3 weld ties per seam.
4) All two sided tests, (except Test #2), were connected to the loading beam and restrained end.

not show any significant change. This orientation behavior is attributed to the fact that for untopped slabs, oriented in the weak direction, the seam fails prematurely before the planks have attained their full capacity; thereafter, the planks act separately leading to a reduction in the peak load. For the topped slabs, the topping prevents the seam failure; therefore forcing the planks to act as one diaphragm, as in the case more like an ordinary reinforced concrete diaphragm.

The addition of the two-inch topping to the planks increased the diaphragm action and produced a stiffer diaphragm. The largest change in the FME load was attained for slabs connected on all four sides. The weak orientation (EW - planks oriented parallel to the applied load) exhibited a bigger increase in the FME and peak loads compared with the increase of the strong orientation (NS). Also, the planks with four sides connected to the loading frame experienced a larger increase than those with only two sides connected.

Increasing the thickness of the planks produced larger effects for the weak orientation, while a less noticeable change was recorded for the strong orientation. The increase of the number weld ties produced a stronger diaphragm, especially for the peak load level.

In order to establish the shear and tension strength of the seams, a series of elemental tests were performed. The results of these tests were reported to the Third Meeting of the U.S.-Japan Joint Technical Coordinating Committee on Masonry Research.

An analytical research on the development of a hysteretic model for hollow-core plank diaphragms is being conducted at Iowa State University as part of TCCMAR research program (Task 2.4a) [348].

4. INITIAL STIFFNESS AND MAXIMUM LOAD PREDICTION

4.1 General

A complete description of the behavioral characteristics of a structure throughout the elastic and plastic ranges can be obtained with a hysteretic model. This type of model predicts the force-displacement relation for a system utilizing stiffness and strength information. This chapter addresses the equations required to predict the initial stiffness and peak strength for steel deck reinforced concrete (SDRC), cold-formed steel, precast prestressed, and plywood diaphragms.

4.2 Cold-Formed Steel Diaphragms

The following discussion of initial stiffness and peak load prediction was taken from Reference [307]. The design recommendations in the following paragraphs are limited to properly interconnected diaphragm panels having thicknesses to between 0.014 inches and 0.064 inches with panel depths D between the nominal limits of 9/16 inches and 3 inches.

4.2.1 Diaphragm Strength

The shear strength of a diaphragm system can be limited by the strength of the connections, local buckling in the panels, or by general plate-like buckling of the whole diaphragm area.

4.2.1.1 Strength of the Connections

The shear strength of cold-formed steel diaphragm, when controlled by the strength of connections is limited to the smaller value from Equations 4-1, 4-2, or 4-3. Consider the diaphragm in Figure 24 where three panels are shown. The panels may be connected to the support frame by structural connections, having a strength Q_f, along any structural member. Stitch or sidelap connections, having a strength Q_s, may be installed along the dashed lines to form sheet-to-sheet connections away from cross supports.

<u>Edge Fasteners</u>

Figure 25 represents the edgemost half panel and the forces transferred to it from P as the ultimate value P_u is approached. For edge fasteners the strength of the diaphragm is given by

$$S_u = P_u/L = (2\alpha_1 + n_p\alpha_2 + n_e)Q_f/L \qquad (4\text{-}1)$$

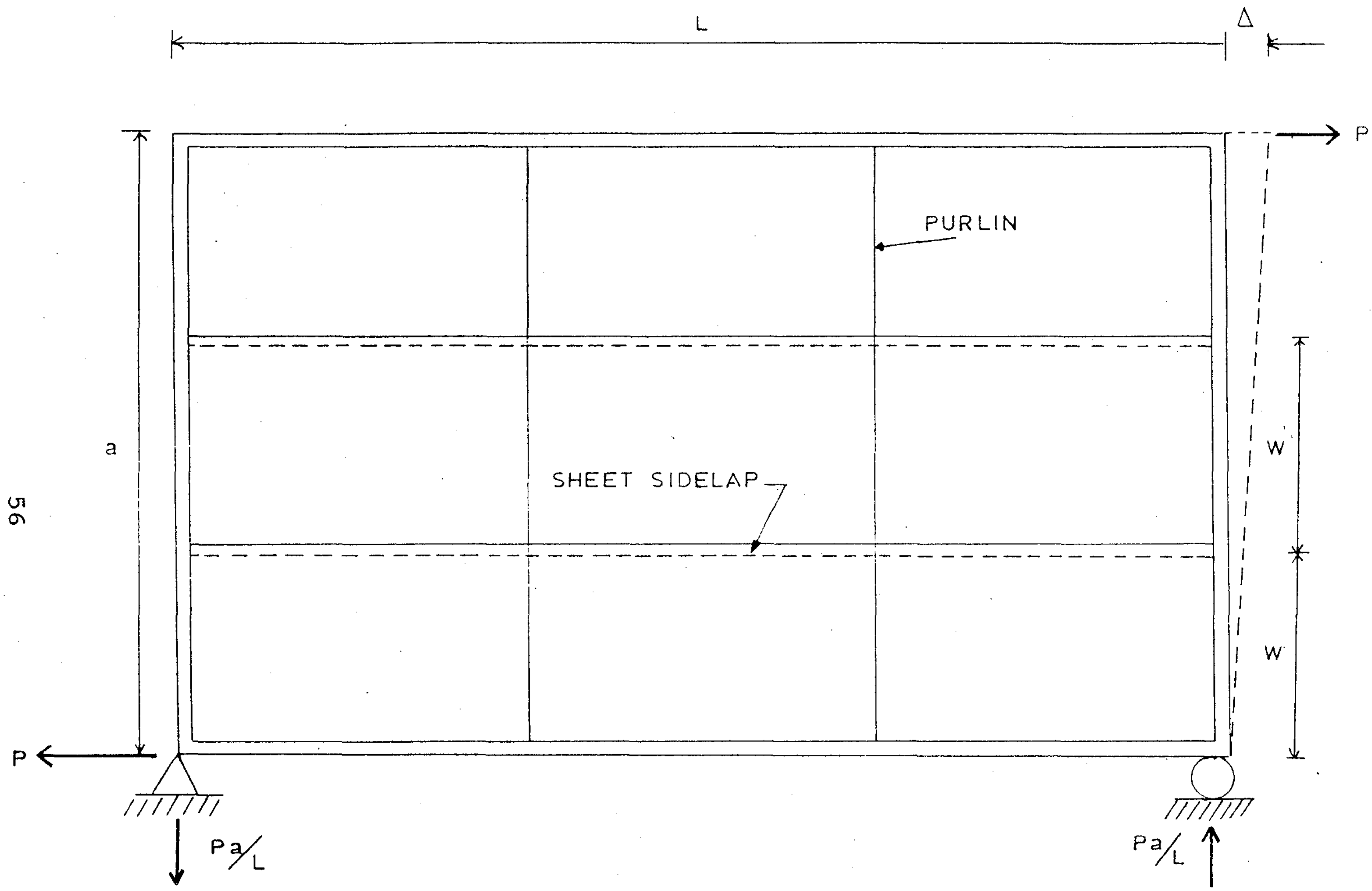

Figure 24. Schematic layout for diaphragm [307]

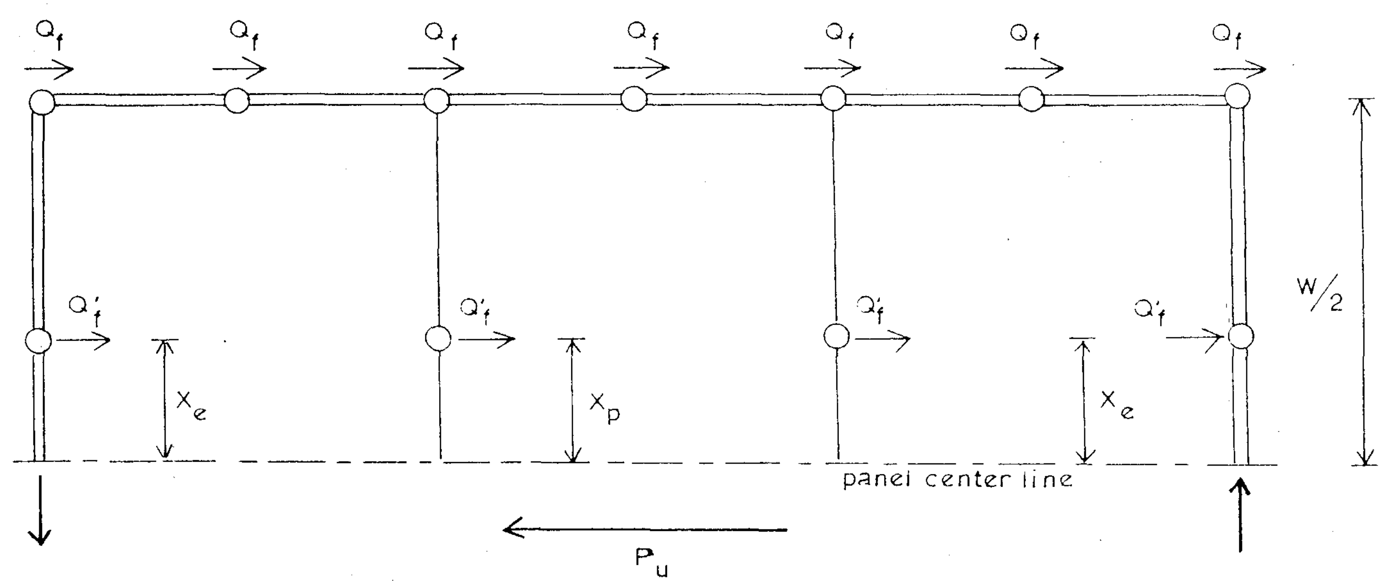

Figure 25. Outer edge panel [307]

where

P_u = diaphragm strength, kips

S_u = ultimate shear, kips/ft.

n_e = number of edge connectors between cross supports (n_e = 3 in Figure 25)

α_1 = $(1/w)\Sigma x_e$, end distribution factor with summation across full width w

x_e = distance from panel centerline to any fastener in a panel at the end support, in.

w = panel width, in.

α_2 = purlin dist. factor similar to α_1

L = panel length, ft.

n_p = number of purlins excluding those at ends or end laps where connection patterns may differ.

Q_f = structural fastener strength, kips (see Section 4, Reference [307])

Interior Panels

Figure 26 shows a free body of an interior panel where Q_s represents a sidelap (stitch) connector and $P_u w/L$ is the transferred end-member axial force. For interior panels the strength of the diaphragm is given by

$$S_u = P_u/L = [2(\lambda-1) + n_s\alpha_s + (1/w^2)(2n_p\Sigma x_p^2 + 4\Sigma x_e^2)]Q_f/L \quad (4\text{-}2)$$

where

Q_s = stitch connector strength, kips (see Section 4, Reference [307])

n_s = number of stitch connectors within the length L.

α_s = Q_s/Q_f

λ = $1-DL_v/[240(t)^{0.5}]$

D = panel depth, in.

L_v = purlin spacing, ft.

A = 2 for double edge fasteners.

A = 1 for single fasteners at panel edges.

L = panel length, ft.

End Members

The fastener at panel corners limit S_u to:

$$S_u = [N^2B^2/(L^2N^2 + B^2)]^{0.5}Q_f \quad (4\text{-}3)$$

where

N = number of fasteners per foot along ends.

B = $n_s\alpha_s + (1/w^2)(2n_p\Sigma x_p^2 + 4\Sigma x_e^2)$

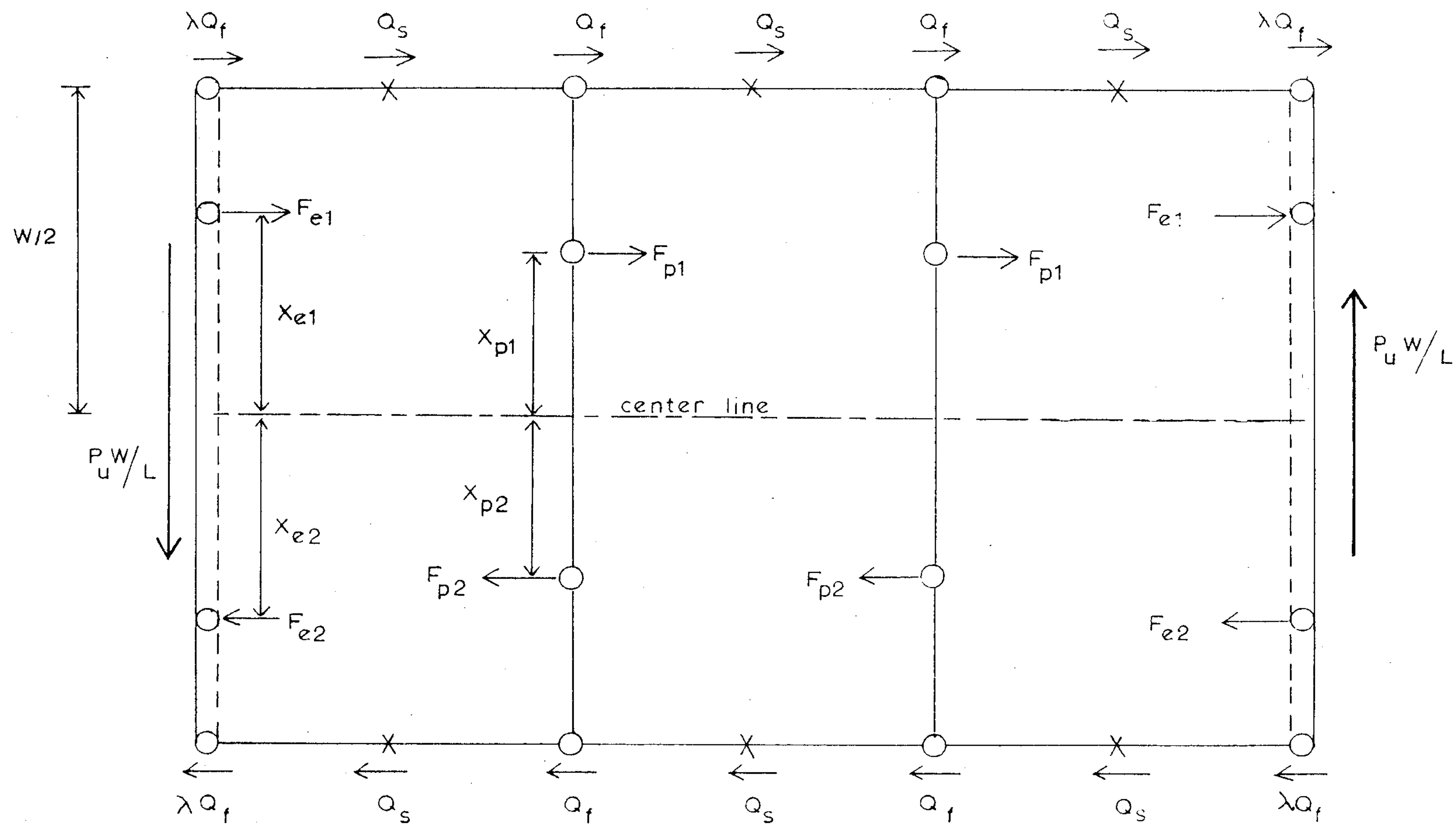

Figure 26. Interior panel force distribution [307]

4.2.1.2 Strength Controlled by Panel Buckling

The probability of plate-like shear buckling, is small for most common installations. For relatively ideal corrugated diaphragms, Easley [139] has presented an approach to the critical shear load. That approach is modified at Reference [307], conservatively treating the limiting case as being controlled by two end spans, resulting in a critical load of:

$$S_c = (12.95X10^3/L^2)(I^3t^3d/s)^{0.25} \tag{4-4}$$

where

S_c = critical load, kips/ft
I = panel moment of inertia, in.^{4}/ft. of width.
d = corrugation pitch, in.
s = developed flute width = $2(e + w) + f$, in.
L = design length, ft.

4.2.2 Diaphragm Stiffness

The stiffness of cold-formed steel deck diaphragms can be obtained with the following equation

$$G' = E\ t/[2(1 + \upsilon)s/d + \varphi D_n + C] \tag{4-5}$$

where

E = modulus of elasticity, kips/in^2
υ = Poisson's ratio, 0.3
D_n = warping constant (see Appendix IV, Reference [307])
s = girth of corrugation per rib, in.
d = corrugation pitch, in.
t = base metal thickness, in.
φ = purlin effect factor on warping (see Table 3.3-2, Reference [307]).
C = connector slip parameter.

$$C = (E\ t/w)S_f[24L/(2\alpha_1 + n_p\alpha_2 + 2n_sS_f/S_s)] \tag{4-6}$$

where

w = panel width, in.
S_f = structural connection flexibility, in/kip
S_s = sidelap connection flexibility, in/kip
L = panel length, feet.
E = modulus of elasticity, 29500 kips/in^2

4.3 Steel Deck Reinforced Concrete Diaphragms

The following discussion of initial stiffness and peak strength prediction was taken from Reference [378]. In this Reference equations were developed to predict the initial stiffness and ultimate strength of SDRC diaphragms. These equations were based on an edge concept, which considered the force to be transferred from the load frame into the diaphragm within a relatively narrow region adjacent to the framing members. Elastic edge force distributions were used to determine the initial stiffness expressions and inelastic edge force distributions were used to determine the ultimate strength expressions.

4.3.1 Diaphragm Strength

The strength of a SDRC diaphragm was limited by one of the three primary failure modes which are a diagonal tension failure of the concrete, a failure of the edge fasteners or a failure of the shear transfer mechanism. The limiting predictive capacity of the diaphragm will be taken as the minimum value from Equations 4-7, 4-21, or 4-22.

4.3.1.1 Diagonal Tension

The strength of SDRC diaphragm, when controlled by diagonal tension failure of the concrete, was determined in Reference [390] by using the ACI [524] shear wall equation. For application to SDRC diaphragms the tensile strength was taken as $4\sqrt{f'_c}$ and the effective depth was taken as the total depth minus two times the edge zone distance which could be taken conservatively as 80 percentage of the depth. Thus the equation is

$$V = 3.2 t_e b \sqrt{f'_c} \qquad (4\text{-}7)$$

where

V = shear strength of composite diaphragm, lbs.
t_e = $t_c + n_s t_s (d/s)$ = effective composite web thickness, in.
t_c = average concrete thickness considering ribbed geometry, in.
n_s = shear modular ratio of steel deck to concrete.
t_s = steel deck thickness, in.
d = corrugation spacing, in.
s = developed width around one corrugation, in.
f'_c = concrete compressive strength, psi.
b = depth of diaphragm, in.

4.3.1.2 Edge Fasteners

The capacity of a SDRC diaphragm based on a fastener failure utilized the edge force distributions obtained from a finite element analysis as shown in Figure 27 (the reader is referred to Reference [378] for a more complete discussion regarding the derivations.) Since different forces typically act along each edge, which may have different number of fasteners, the strength

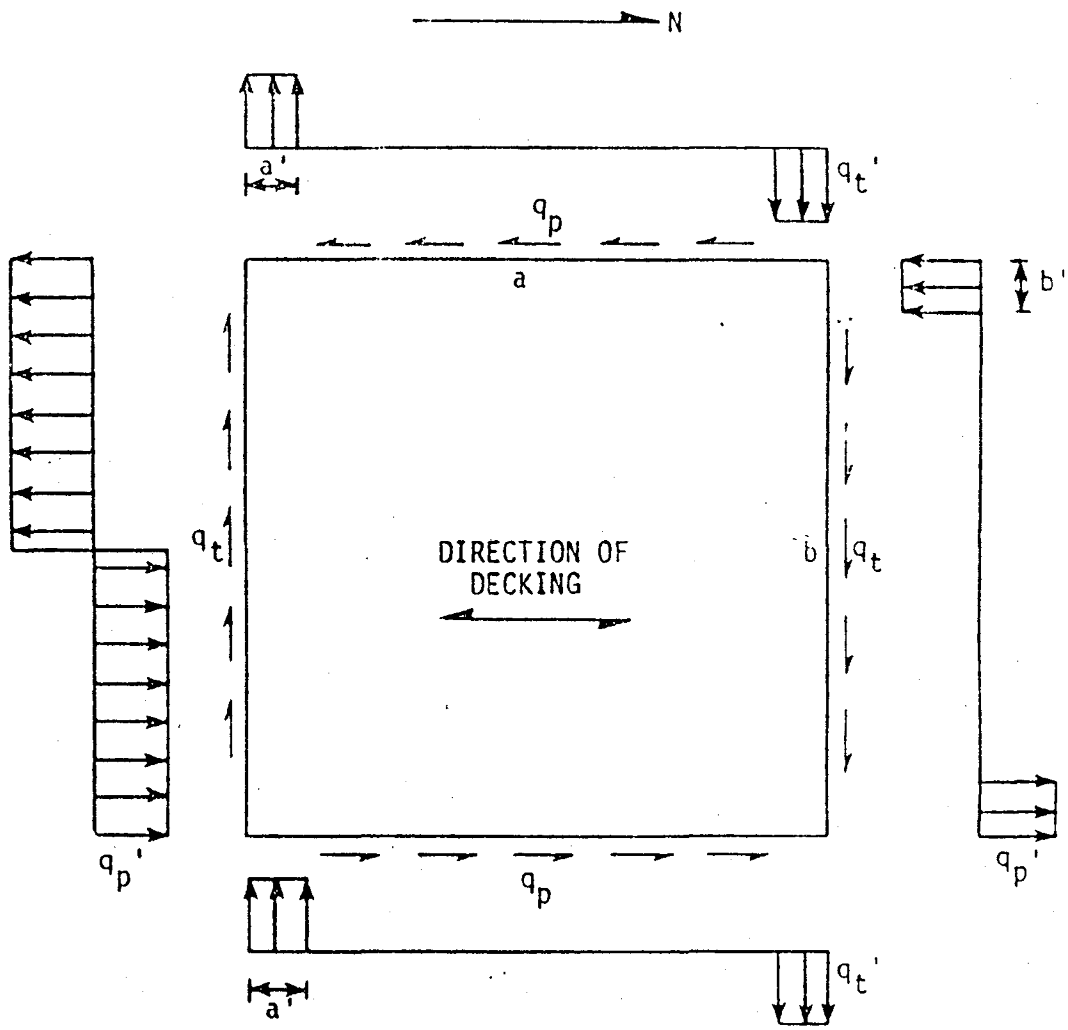

Figure 27. Assumed forces acting on edges [378]

based on the fasteners along either of the two sides must be checked.

Considering fastener "A" in Figure 28, the strength is given by

$$V = 1/\{[b/(b + \alpha l_t')/(n_b Q_{ut})]^2 + [b/(\beta b + l_p')/(n_b Q_{up}')]^2\}^{0.5} \quad (4\text{-}8)$$

where

b = depth of diaphragm, in.

$$\alpha = q_t'/q_t = (0.87k_t/k_t' + 0.73)^{-1} \quad (4\text{-}9)$$

$$\beta = q_p/q_p' = 0.81k_p/k_p' + 0.23 \quad (4\text{-}10)$$

q_t, q_t', q_p, q_p' = edge forces

l_t' = $2a' - 2a'^2/a$

l_p' = $(b^2 + 4bb' - 4b'^2)/4a$

n_b = number of fasteners along b.

Q_{ut} = strength of fastener in direction of q_t.

Q_{up}' = strength of fastener in direction of q_p'.

For diaphragms welded according to the guidelines of References [525,526,527], the values of α and β are recommended as follows:

$$\alpha = 0.64 \qquad \beta = 1.32$$

when headed shear studs are used as fasteners, Equations (4-9) and (4-10) should be used to determine α and β. In Equations (4-9) and (4-10) the edge zone stiffness can be evaluated using the following Equation [130]:

$$k = 145.3\ Q_{su}/S_s \quad (4\text{-}11)$$

where

k = edge zone stiffness, kips/in.

Q_{su} = stud fastener capacity in load direction, kips

S_s = stud spacing, in.

The values recommended for a' and b' are as follows:

welded diaphragms: $a' = b' = 5.83 I_y^{0.35} - 2$

studded diaphragms: $a' = b' = 15$ inches.

where

I_y = moment of inertia about the weak axis of the framing member.

Considering fastener "B" in Figure 28, the strength is given by

$$V = 1/\{[a/(b + \beta l_p')/(n_a Q_{up})]^2 + [a'/(\alpha^{-1} b + l_t')/(n_a' Q_{ut}')]^2\}^{0.5} \quad (4\text{-}12)$$

where

n_a = number of fasteners along a.

n_a' = number of fasteners along a'.

Q_{up} = strength of fastener in the direction of q_p.

Q_{ut}' = strength of fastener in the direction of q_t'.

If there are a sufficient number of fasteners designated "C" in Figure 27, to carry load along side a, after "B" connectors have failed the strength is given by:

$$V = (b + \beta^{-1} l_p')(n_a - 2n_a')Q_{up}/a \tag{4-13}$$

When headed shear studs are used as fasteners the strength were determined with the following expressions [353]:

$$Q_{sol} = 0.00666 A_{st} f'_c{}^{0.3} E_c{}^{0.44} \tag{4-14}$$

where
A_{st} = cross sectional area of stud, in^2
E_c = concrete modulus of elasticity, psi
Q_{sol} = stud strength in solid slab, kips

Equation (4-14) is used for stud strength in a direction parallel with the ribs of the deck. For the strength in a direction perpendicular to the ribs of the deck the following equation is used [525,526]:

$$Q_{rib} = (0.85/\sqrt{N})[(H - h)/h](w/h)Q_{sol} \le Q_{sol} \tag{4-15}$$

where
Q_{rib} = strength of stud in a transverse rib.
N = number of studs in a rib.
H = height of stud after welding.
h = heigth of stud after welding.
w = average rib width.

Where a stud is located near an edge in the direction of an applied shear force a reduction in calculated strength [321] is applied as given by:

$$Q'_{sol} = Q_{sol}[(d_{es} - 1)/(8d_{st})] \le Q_{sol} \tag{4-16}$$

where
d_{es} = distance from the center of the stud to the free edge.
d_{st} = diameter of stud.

When arc spot welds are used as fasteners, the strength, Q_u, is determined using the AISI [528] weld strength equations given by the smaller of:

$$Q_u = 0.625 d_e^2 F_{xx} \tag{4-17}$$

or one of the following

for $d_a/t \le 140/\sqrt{F_u}$ $\quad Q_u = 2.2 t d_a F_u$ (4-18)
for $140/\sqrt{F_u} < d_a/t \le 240/\sqrt{F_u}$
$\quad Q_u = 0.28[1 + 960t/(d_a\sqrt{F_u})] t d_a F_u$ (4-19)
for $d_a/t > 240/\sqrt{F_u}$ $\quad Q_u = 1.4 t d_a F_u$ (4-20)

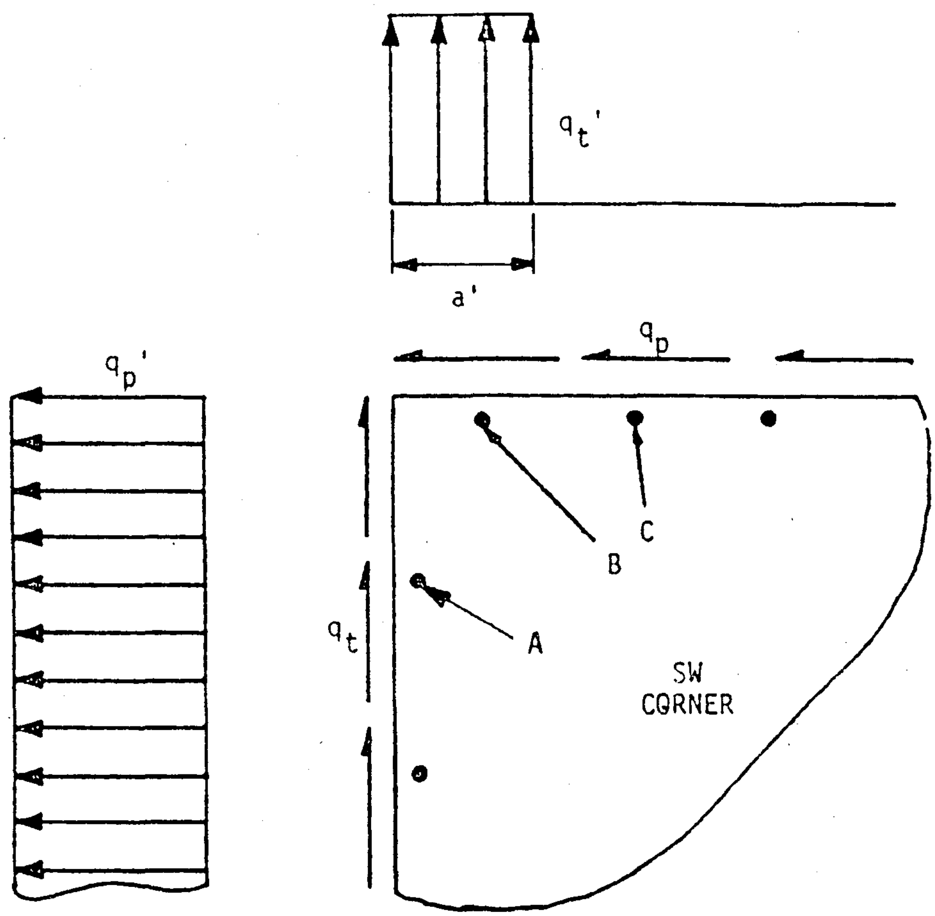

Figure 28. Assumed forces acting on edge fasteners [378]

where

d = weld diameter, in.

d_a = average diameter of arc spot weld at mid-thickness (d-t for single sheet, d-2t for double sheet), in.

d_e = effective diameter of fused area = $0.7d - 1.5t \le 0.55d$, in.

t = thickness of sheet, in.

F_u = ultimate strength of sheet, ksi

F_{xx} = AWS weld designation strength, ksi

The predicted strength of a SDRC diaphragm controlled by a fastener failure is taken as

$$V = \text{Minimum}\{\text{Eqn. 4-8, Maximum[Eqn. 4-12, Eqn. 4-13]}\} \qquad (4\text{-}21)$$

4.3.1.3 Shear Transfer Mechanism

The strength of a SDRC diaphragm, when controlled by a shear transfer mechanism failure is determined using results from the elemental pushoff tests [404]. The strength is given by

$$V = \text{Minimum}\{\text{Eqn. 4-23, Eqn. 4-24}\} \qquad (4\text{-}22)$$

$$V = C_{po}Q_{tpo}(b + \alpha l_t') \qquad (4\text{-}23)$$

$$V = C_{po}Q_{ppo}(b + \beta^{-1} l_p') \qquad (4\text{-}24)$$

where

C_{po} = pushoff test correction factor = $(t_s d/s)n_s + t_c)/t_c$

Q_{tpo} = transverse pushoff test strength.

Q_{ppo} = parallel pushoff test strength.

4.3.2 Initial Stiffness

For determining the initial stiffness, the diaphragm was modeled as a deep beam. The total stiffness was considered to consist of several components as is shown in the following equation

$$K_{tot} = 1/(1/K_b + 1/K_s + 1/K_z + 1/K_f) \qquad (4\text{-}25)$$

where

K_{tot} = total initial stiffness of composite diaphragm.

K_b = bending stiffness of composite diaphragm.

K_s = shear stiffness of composite diaphragm.

K_z = edge zone stiffness of composite diaphragm.

K_f = frame connection stiffness.

4.3.2.1 Bending Stiffness

The bending deflection term was based on the assumption that the composite slab act as the web in the deep beam model and the edge beams act as the flanges.

The bending stiffness is given by

$$K_b = 3(E_c I_c + E_b I_b)/a^3 \tag{4-26}$$

where

a = span at the diaphragm.
E_c = concrete modulus of elasticity.
I_c = moment of inertia of composite web.
E_b = moment of inertia of edge beams about "deep beam" neutral axis.

4.3.2.2 Shear Stiffness

Shear deflection for steel deck diaphragms was given by Lutrell [306]. In the shear stiffness evaluation only the web of the deep beam model is assumed effective in resisting shear deformation. The shear stiffness is given by

$$K_s = b[G_s t_s (d/s) + G_c t_c]/a \tag{4-27}$$

where

b = depth of diaphragm.
G_s = shear modulus of elasticity.
G_c = shear modulus of concrete.

4.3.2.3 Edge Zone Stiffness

The edge zone stiffness is given by the following equation:

$$K_z = \{2/[k_t(b + \alpha l_t)] + 2a/[k_p b(b + \beta^{-1} l_p)]\}^{-1} \tag{4-28}$$

where

$l_t = a' - 2a'^2/3a$
$l_p = (b^2 + 3bb' - 2b'^2)/6a$

for welded connections using the results from elemental pushoff test k_t and k_p become

$k_t = C_{po} Q_{tpo}$
$k_p = C_{po} Q_{ppo}$

for studded fasteners use Equation (4-11).

4.3.2.4 Frame Connection Stiffness

The last component of the initial stiffness of the diaphragm be considered is the stiffness of the test frame support connection at the abutment. This rigid body rotation was originally deemed negligible but subsequent monitoring of test specimens revealed that indeed there was a small, yet measurable deflection. Measurements indicated that for the test frame constructed with the W24X76 sections the stiffness k_f was approximately 10,000 kips/in. and for the test frame constructed with the W14X22 sections the stiffness was approximately 4250 kips/in.

4.4 Precast Prestressed Diaphragms

The following discussion of initial stiffness and peak strength prediction was taken from Reference [376]. The strength and stiffness predictions for the hollow-core plank systems were based in the assumption that the connections between the planks and the testing frame would not fail before the diaphragm reached its limit state. Thus, by determination of the distribution of forces in the edge zone, the forces throughout the diaphragm were studied.

4.4.1 Initial Stiffness

The initial stiffness calculations were based on a predictive method initially developed by Porter and Greimann [390], later modified by Dodd [130], and Porter and Easterling [378]. The total deflection was assumed to consist of four components: Bending deflection, shear deflection, edge zone deflection, and deflection due to axial flexibility of edge beam framing connection. The initial stiffness is given by

$$K_{tot} = 1/(1/K_b + 1/K_s + 1/K_z + 1/K_f) \tag{4-29}$$

where

K_{tot} = total diaphragm stiffness, kips/in
K_b = bending stiffness component, kips/in
K_s = shear stiffness component, kips/in
K_z = edge zone stiffness component, kips/in
K_f = frame connection component, kips/in

4.4.1.1 Bending Stiffness

For bending, the diaphragm was considered to be a cantilever girder with the hollow-core planks acting as the web and the edge beam acting as the flanges.

The bending stiffness is given by

$$K_b = 3(E_cI_c + E_bI_b)/a^3 \qquad (4\text{-}30)$$

where

K_b = bending stiffness of system, kips/in
a = length of cantilever girder, in.
E_c = concrete modulus of elasticity of the plank, ksi
I_c = moment of inertia of web, in^4
E_b = modulus of elasticity of edge beam, ksi
I_b = moment of inertia of edge beams about girder neutral axis, in^4

4.4.1.2 Shear Stiffness

For shear stiffness evaluation, only the web is assumed effective against shear and is given by

$$K_s = bG_ct_c/a \qquad (4\text{-}31)$$

where

K_s = shear stiffness of system, kips/in
a = length of cantilever girder, in.
b = depth of cantilever girder, in.
G_c = shear modulus of concrete, ksi
t_c = average thickness of concrete, in.

4.4.1.3 Edge Zone Stiffness

The deflection of the system due to edge zone deformation was based on a idealized edge zone force distribution. The edge zone stiffness is given by

$$K_z = 1/\{(r_3 + r_4)/[k_t(b + l_t")] + 2ar_1/[k_pb(b + l_p")]\} \qquad (4\text{-}32)$$

where

K_z = edge zone stiffness, kips/in
a = length of cantilever girder, in.
b = depth of cantilever girder, in.
k_t, k_p = equivalent edge zone spring stiffnesses determined by using

$$k_{eq} = 145.3Q_{su}/S_s \qquad (4\text{-}33)$$

k_{eq} = equivalent stiffness k_t or k_p, kips/in
S_s = stud spacing, in.
Q_{su} = stud connector capacity in the load direction, kips

$$Q_{su} = 6.66X10^{-3}A_sf'_c{}^{0.3}E_c{}^{0.44} \leq 0.9A_sf_s \qquad (4\text{-}34)$$

A_s = area of stud, in^2
E_c = modulus of elasticity of concrete, psi
f_s = yield strength of the stud, ksi
f'_c = compressive strength of the concrete, psi

$$r_1 = g_p a \coth(g_p a) \qquad (4\text{-}35)$$

$$r_2 = g_p a \,\mathrm{csch}(g_p a) \qquad (4\text{-}36)$$

$$r_3 = g_t (b/2) \coth(g_t b/2) \qquad (4\text{-}37)$$

$$r_4 = [b + r_3 a''(2 - a''/3a)]/(b + a'') \qquad (4\text{-}38)$$

$$l_t'' = a''[r_3(3a - a'') - a'' r_4]/3a \qquad (4\text{-}39)$$

$$l_p'' = (r_1 b^2 + 3 r_2 b b'' - 2 r_2 b''^2)/6a \qquad (4\text{-}40)$$

$$g_p = \sqrt{(k_p/E_b A_b)} \qquad (4\text{-}41)$$

$$g_t = \sqrt{(k_t/E_b A_b)} \qquad (4\text{-}42)$$

A_b = area of edge beam, in^2
E_b = modulus of elasticity of edge beam, ksi
a'' = b'' = initial effective edge zone width = 6 inches

4.4.1.4 Frame Connection Stiffness

The final component of the initial diaphragm deflection was the axial flexibility of the edge beam abutment connections. This frame connection stiffness component served as a correction due to framing and connection movements. The frame connection stiffness has been experimentally determined to be approximately 10,000 kips/in.

4.4.2 Peak Strength

The FME load capacity of the plank diaphragm systems was limited by one of three major categorical failure modes: shear-bond seam failure, tensile-bond seam failure, or diagonal tension failure. Limit states were identified as one of three potential events: initial weld tie shear failure, initial weld tie tension failure, or initial diagonal tension crack. The limiting predictive capacity of the diaphragm will be taken as follows:
a) Peak strength in planks transverse to load direction:

V = Minimum { Maximum [Eqn.(4-52),Eqn.(4-54)], Eqn.(4-70) }

b) Peak strength in planks parallel to load direction:

V = Minimum { Maximum [Eqn.(4-61),Eqn.(4-66)], Eqn.(4-70) }

In order to predict the FME and limit state strength the following equations are needed

$$q_{pav} = V/(b + l_p") \quad (4-43)$$

$$q_{p1} = q_{pav}ag_p/\tanh(g_pa) \quad (4-44)$$

$$q_{p2} = q_{p1}\text{sech}(g_pa) \quad (4-45)$$

$$q_{tfav} = V/(b + l_t") \quad (4-46)$$

$$q_{tfo} = q_{tfav}bg_t/[2\sinh(g_tb/2)] \quad (4-47)$$

$$q_{tf1} = q_{tfo}\cosh(g_tb/2) \quad (4-48)$$

$$q_{tb} = r_4q_{tfav} \quad (4-49)$$

$$Q_{tf} = (q_{tfo}/g_t)[\sinh(g_tb/2) - \sinh(48g_t)] \quad (4-50)$$

$$Q_p = (q_{p2}/g_p)\{\sinh[g_p(42-a)] - \sinh(-g_pa)\} \quad (4-51)$$

4.4.2.1 FME and Limit State prediction for Shear-Bond Failure (planks transverse to load direction)

FME prediction for shear-bond failure (planks transverse to load direction) is given by

$$V^p_{fme} = [T_{av}'d_pl_s + \gamma d_tl_s](b + l_p")/(ar_5) \quad (4-52)$$

where

T_{av}' = 69 psi = reduced shear stress
d_p = Grout depth of penetration, in
l_s = bond length, usually equal to a, in
d_t = depth of topping, in
f'_{ct} = topping concrete compressive strength, psi
γ = $0.2f'_c \le 800$ psi

$$r_5 = 1 + g_p\text{sech}(g_pa)b"/[2\tanh(g_pa)] + g_p(b^2/4 - 48^2)/[b\tanh(g_pa)] \quad (4-53)$$

Limit state prediction for shear-bond failure (planks transverse to load direction) is given by

$$V^p_{ls} = [5.5n + 0.9(N_c + N_t) + \gamma d_tl_s](b + l_p')/(a + b/2 - 42) \quad (4-54)$$

where

$l_p' = (b^2 + 4bb' - 4b'^2)/4a$ (4-55)
n = number of weld ties.

$$N_t = [r_7/(a - l_c/3) - r_6][1 - l_t/(3a - l_c)] \quad (4-56)$$

$$N_c = (N_t l_t/3 + r_7)/(a - l_c/3) \quad (4\text{-}57)$$

$$r_6 = q_{tb}(b/2 - 48) + (q_{tb} - q_{tf1})a''/2 - Q_{tf} \quad (4\text{-}58)$$

$$rr_7 = a(b/2 - 48)(q_{tb} - q_{pav}) - (q_{tb} + q_{tf1})a''^2/6 + q_{tb}aa''/2 \quad (4\text{-}59)$$

$$r_7 = rr_7 - (3b/2 - b'' - 144)q_{p2}b''/6 - q_{p1}(b/2 - 48)^2(16/b + 1/3) \quad (4\text{-}60)$$

$$l_c = l_t = a/2$$

4.4.2.2 FME and Limit State Prediction for Tensile-Bond Failure (planks parallel to load direction)

FME prediction for tensile-bond failure (planks parallel to load direction) is given by

$$V^p_{fme} = \text{Minimum}\{\ V_1 \text{ Eqn.}(4\text{-}62),\ V_2 \text{ Eqn.}(4\text{-}63)\} \quad (4\text{-}61)$$

$$V_1 = \nu'_{tav}d_p l_t(3b - l_t - l_c)/(-3r_8) + 5\sqrt{(f'_{ct})}d_t l_s \quad (4\text{-}62)$$

$$V_2 = (2/3)[T'_{av}d_p l_s + \gamma d_t l_s](b + l_t'')/[r_4(b + a'')] \quad (4\text{-}63)$$

where

$$\nu'_{tav} = 40 \text{ psi} = \text{reduced tensile stress}$$

$$rr_8 = bg_p/6 + \text{sech}(g_p a)\{\sinh[g_p(42 - a)] + \sinh(g_p a)\} \quad (4\text{-}64)$$

$$r_8 = r_4(42a'' - a''^2/3 + 42b)/(b + l_t'') - ab/[(b + l_p'')\tanh(g_p a)]\}rr_8 \quad (4\text{-}65)$$

Limit state prediction for tensile-bond failure (planks parallel to load direction) is given by

$$V^p_{ls} = \text{Minimum}\{\ V_3 \text{ Eqn.}(4\text{-}67),\ V_4 \text{ Eqn.}(4\text{-}68)\ \} \quad (4\text{-}66)$$

$$V_3 = [5.5n + 0.9(N_c + N_t) + \gamma d_t l_s](b + l_t')/(b + 2a') \quad (4\text{-}67)$$

$$V_4 = (b - l_c/3 - l_t/3)[N_c + 5\sqrt{(f'_{ct})}d_t l_s][(b^2/4 + 42b)/(b + l_p') - (84a' - a'^2 + 42b)/(b + l_t')] \quad (4\text{-}68)$$

where

$$l_t' = 2a'' - 2a''^2/2 \quad (4\text{-}69)$$

4.4.2.3 FME and Limit State Prediction for Diagonal Tension Failure

The diagonal tension failure represented an upper limit for a concrete diaphragm. Diagonal tension failure calculations are based on Equation (11-32) from the American Concrete Institute 318-83 code [524],

$$V_c = 3.3\sqrt{(f'_c)}bd + N_{cp}d/(4l_w) \qquad (4\text{-}70)$$

where
- V_c = diagonal shear capacity of the concrete.
- f'_c = plank concrete compressive strength.
- b = diaphragm width.
- d = effective plank depth.
- N_{cp} = normal compressive force (prestressing)
- l_w = 0.8b

4.5 Plywood Diaphragms

In wood structures, the roof and floor sheathing are designed for gravity loads, but they can also carry lateral load. The use of roof and floors as diaphragms in structures basically require no modification to the basic construction. Common types of wood diaphragms are plywood diaphragms, heavy timber decking, diagonal sheathing, double diagonal sheathing, and transverse sheathing. In new construction the most common of the various sheathing materials is plywood because of a combination of factors including low installation cost, excellent rigidity and strength.

Resistance of the sheathing to shear caused by lateral loads depends on four things: sheathing thickness and layout, nailing type and spacing, provisions for blocking, and width of framing members. Deflection of plywood diaphragms can be evaluated considering the usual bending and shear components as well as other factors, such as joint slip in the chords, and nail deformation.

4.5.1 Diaphragm Strength

A plywood diaphragm may fail by nail heads pulling through the panel face, by nails pulling out through panel edges, by nails causing framing members to split, or by buckling of the plywood. Generally, shear-through-the-thickness failure is not a factor.

Table No. 25-J-1 [531], shows allowable shears in horizontal plywood diaphragms as a function of the sheathing thickness and layout, nailing type and spacing, provisions for blocking, and width of framing members. The allowable shears from Table No. 25-J-1 are based on extensive tests results - see Appendix A from Reference 477 - and using a factor of safety against ultimate strength of about 3 to 4. See also Table No. 25-K-1 [531] for allowable shears in plywood shearwalls.

For the cases not covered by Table No. 25-J-1, the allowable shear in blocked diaphragms is calculated as the smaller of (1) the allowable shear, V_{cp}, based in plywood shear stress and (2) the allowable shear, V_{np}, based on lateral fastener load at the boundary. V_{cp} and V_{np} are evaluated as follows [477]:

$$V_{cp} = \alpha\ \beta\ \gamma\ V \tag{4-71}$$

where

α = a factor of 1.33 for load duration (assuming wind or seismic load)

β = a factor of 12 (in/ft)

γ = the effective thickness for shear in the plywood (Table 1 of Reference 529)

V = allowable stress for shear through the thickness of the plywood (Table 3 of Reference 529)

$$V_{np} = \delta\ \epsilon\ \zeta\ \eta\ \theta\ \iota\ \kappa\ \lambda\ V_n \tag{4-72}$$

where

δ = a factor of 1.30 for diaphragm construction by NDS Section 8.8.5.5 [530]

ϵ = a factor of 1.33 for load duration

ζ = number of nails per foot, except use 2.25 nails per foot (experimentally derived) when there are actually 2 nails per foot.

η = a factor of 0.89 if 2-in. nominal lumber is used, or if two rows of fasteners are used in 3-in. nominal lumber, or if three rows are used in 4-in. nominal lumber.

θ = a factor of 0.90 if the diaphragm consists of Douglas fir framing members and non-structural-I plywood.

ι = a factor of 0.85 if nails are spaced 2 in. c/c at boundary.

κ = for 10d nails only, an additional factor of 0.85 if nails are spaced 2 in. or 2.5 in. c/c at boundary when boundary members are single 2-in. nominal members.

λ = for non-Douglas fir lumber, a factor of 0.82 for lumber group III and a factor of 0.65 for lumber group IV.

V_n = lateral load design value for the particular nail [530]

4.5.2 Diaphragm Stiffness

The deflection of a uniformly loaded rectangular blocked plywood diaphragm can be evaluated with the following formula [477]:

$$\Delta = 5vL^3/(8EAd) + vL/(4Gt) + 0.188Le_n + (\Sigma\Delta_c x_s)/2d \tag{4-73}$$

where

Δ = the calculated deflection at the centerline, in.
v = maximum shear due to design loads in direction under consideration, lb/ft
L = diaphragm length, ft.
d = diaphragm depth, ft.
E = elastic modulus of the flange material, psi
A = area of flange cross section, in^2.
G = modulus of rigidity of plywood, psi
t = effective thickness of plywood for shear, in.
e_n = nail deformation for a given load on a nail (see Table B-4 in Reference 477), in.
x_s = distance to the splice from the support, ft.
Δ_c = individual chord splice slip, in.

The deflection of a blocked plywood shearwall may be calculated by use of the following formula:

$$\Delta = 8vh^3/(EAb) + vh/(Gt) + 0.376he_n + d_a \qquad (4\text{-}74)$$

where

Δ = the calculated deflection at the top of the wall, in.
v = maximum shear due to design loads at the top of the wall, lb/ft.
A = area of boundary elements cross section (vertical member at shearwall boundary), in^2.
h = wall heigth, ft.
b = wall width, ft.
d_a = deflection due to anchorage details, in.
E = elastic modulus of boundary element (vertical member at shearwall boundary), psi

The initial stiffness, K, can be derived from Equation (4-73) or Equation (4-74) by differentiating Δ as follows:

$$K = [\ (\ d\Delta/dV\ )^{-1}\ ]_{V=0} \qquad (4\text{-}75)$$

where

K = initial stiffness, lb/in.
Δ = diaphragm or shearwall deflection, in.
V = maximum shear due to design loads, lb.

5. SUMMARY

The research undertaken for this project is part of the U.S.-Japan Coordinated Program for Masonry Building Research. Each category of this program is conducted under the supervision of the Technical Coordinating Committee for Masonry Research (TCCMAR). The Study of floor diaphragms, which is the fifth research task was divided into separate tasks. The objective of this project (Task 5.2) was the collection of existing literature and data generated from the discussion and testing of horizontal diaphragms.

As part of this study previous diaphragm research was reviewed. The review included Cold-Formed Steel, Composite Steel Deck, Wood, Reinforced Concrete, and Precast Concrete diaphragms. Data generated from diaphragm testing is also included for the following diaphragms types: Planks, Steel-Deck-Reinforced diaphragms, Wood Diaphragms, and Steel Deck on wood framed diaphragms.

TCCMAR has requested that the hysteretic model named EKEH model which is part of the Lumped Parameter Model, be considered as part of the overall project. In order to assemble existing experimental data of floor diaphragms and reducing to a form required for static and dynamic analysis (e.g. EKEH model), the following data is needed: maximum deformation, deformation at peak strength, peak strength, initial stiffness, and post peak strength lower limit factor. Only some of this parameters were found on the experimental data generated from previous work. Additional work is needed to provide the complete set of data required for EKEH model, principally on Wood Diaphragms.

Additional work on the development of a hysteretic model for hollow-core planks is being conducted at Iowa State University as part of TCCMAR research program (Task 2.4a) [348].

6. ACKNOWLEDGEMENTS

This research investigation was sponsored by the National Science Foundation (Grant No. CES - 8517028, Formerly Grant No. CEE - 8517028). The work was conducted at Iowa State University through the auspices of the Engineering Research Institute.

The authors wish to thank the contributions made by the following graduate students: Ron J. Meyer, Paul M. Tremel, and Aziz A. Sabri.

The authors are greatly appreciative of the work done by Terri Smith and Karen Gilbertson who aided in the data collection for the research.

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8. APPENDIX A: DIAPHRAGM DATA

8.1 Cold-Formed Steel Deck

8.1.1 Data obtained from Reference 25, 305, and 306

TABLE A1. WELDED DIAPHRAGM TEST DATA

TEST #	SHEAR STIFFNESS K/in.	SHEAR STRENGTH plf	DISPL. @ PEAK IN.	FAILURE MODE
W-1	15.2	624	NR	NR
W-2	18.4	605	NR	NR
W-3	10.6	320	NR	NR
W-4	14.1	650	NR	NR
W-5	11.1	401	NR	NR
W-6	12.0	461	NR	NR
W-7	16.2	508	NR	NR
W-8	12.1	510	NR	NR
W-9	9.4	467	NR	NR
W-10	40.3	1040	NR	NR
W-11	21.6	640	NR	NR
W-12	20.7	630	NR	NR
W-13	20.6	580	NR	NR
W-14	15.9	560	NR	NR
W-15	16.9	450	NR	NR
W-16	23.2	505	NR	NR
W-17	14.4	339	NR	NR
W-18	21.3	830	NR	NR
W-19	23.0	807	NR	NR
W-20	24.9	912	NR	NR
W-21	14.2	790	NR	NR
W-22	67.7	995	NR	NR
W-23	87.2	1090	NR	NR
WB-1	32.3	1260	NR	NR
WB-2	12.6	495	NR	NR
WB-3	11.8	480	NR	NR
WB-4	16.5	775	NR	NR
WB-5	16.0	525	NR	NR
WB-6	16.6	580	NR	NR
WB-7	10.6	492	NR	NR
WB-8	12.7	615	NR	NR
WB-9	11.5	423	NR	NR
WB-10	8.3	311	NR	NR
WB-11	12.8	339	NR	NR
WB-12	32.4	1577	NR	NR
WB-13	19.4	1125	NR	NR
WB-14	14.4	719	NR	NR
WB-15	13.8	694	NR	NR

TABLE A1 CONTINUATION

TEST #	SHEAR STIFFNESS K/in.	SHEAR STRENGTH plf	DISPL. @ PEAK IN.	FAILURE MODE
WB-16	8.4	498	NR	NR
WB-17	19.8	580	NR	NR
WB-18	22.7	906	NR	NR
WB-19	34.5	1400	NR	NR
WB-20	32.0	1500	NR	NR
WB-21	19.0	1025	NR	NR
WB-22	19.0	950	NR	NR
WB-23	13.6	720	NR	NR
WB-24	13.5	783	NR	NR
WB-25	27.8	1140	NR	NR
WB-26	26.3	1295	NR	NR
WB-27	14.8	750	NR	NR
WB-28	14.9	730	NR	NR
WB-29	10.1	540	NR	NR
WB-30	8.3	530	NR	NR
WB-31	27.4	1250	NR	NR
WB-32	12.3	472	NR	NR
WB-33	29.8	1400	NR	NR
WB-34	23.0	1000	NR	NR
WB-35	31.2	1400	NR	NR
WB-36	15.4	348	NR	NR
WB-37	12.8	409	NR	NR
WB-38	17.6	460	NR	NR
WB-39	31.0	738	NR	NR
WB-40	13.1	696	NR	NR
WB-41	10.2	583	NR	NR
A-1	8.3	339	NR	NR
A-2	10.1	432	NR	NR
A-3	8.2	265	NR	NR
A-4	11.7	240	NR	NR
A-5	11.9	293	NR	NR
A-6	13.3	396	NR	NR
A-7	20.0	479	NR	NR
A-8	7.8	411	NR	NR
A-9	9.1	320	NR	NR
A-10	9.4	316	NR	NR
A-11	15.3	451	NR	NR
A-12	26.1	706	NR	NR
A-13	21.6	591	NR	NR
A-14	6.6	408	NR	NR
A-15	22.4	832	NR	NR
A-16	12.6	565	NR	NR
A-17	6.6	245	NR	NR
A-18	20.9	820	NR	NR
A-19	10.9	365	NR	NR
A-20	13.0	548	NR	NR

TABLE A1 CONTINUATION

TEST #	SHEAR STIFFNESS K/in.	SHEAR STRENGTH plf	DISPL. @ PEAK in.	FAILURE MODE
A-21	12.9	565	NR	NR
A-22	14.1	507	NR	NR
A-23	28.8	900	NR	NR
A-24	26.5	750	NR	NR
A-25	6.0	263	NR	NR
A-26	9.3	425	NR	NR
A-27	18.3	575	NR	NR
A-28	34.0	365	NR	NR
A-29	19.7	740	NR	NR
A-30	15.7	550	NR	NR
A-31	10.0	345	NR	NR
A-32	20.1	615	NR	NR
A-33	11.4	515	NR	NR
I-1	9.1	350	NR	NR
I-2	17.5	550	NR	NR
I-3	21.6	755	NR	NR
I-4	16.1	535	NR	NR
I-5	11.3	315	NR	NR
I-6	8.0	275	NR	NR
I-7	12.7	360	NR	NR
I-8	24.8	520	NR	NR
I-9	17.5	500	NR	NR
I-10	36.6	750	NR	NR
1-B[a]	2.9	283	NR	PUDDLE WELD FAILURE
2-B[a]	8.3	438	NR	"
3[a]	3.1	313	NR	"
4[a]	7.6	447	NR	"
5[a]	2.1	438	NR	"
6[a]	2.4	330	NR	"
7[a]	4.4	533	NR	"
8[a]	5.2	675	NR	"
9[a]	5.0	342	NR	"
10[a,b]	---	300	NR	"
11[a]	8.1	475	NR	"
12[a,b]	---	321	NR	"
13[a,b]	---	358	NR	"
15[a,b]	---	358	NR	"
16[a]	4.8	367	NR	"
17[a,b]	---	358	NR	"

NOTES

[a] data obtained from Reference 25

[b] the diaphragm was previously subjected to 5 reversed load cycles at two different load levels ($0.4P_u$ and $0.6P_u$) and then loaded from zero to failure under monotonic load

TABLE A2. SCREWED DIAPHRAGM TEST DATA

TEST #	SHEAR STIFFNESS K/in.	SHEAR STRENGTH plf	DISPL. @ PEAK IN.	FAILURE MODE
I-1	22.37	430	NR	NR
I-2	21.78	650	NR	NR
I-3	16.00	335	NR	NR
I-4	16.67	495	NR	NR
I-5	15.31	388	NR	NR
I-6	22.96	610	NR	NR
W-7	4.80	285	NR	NR
W-8	5.30	275	NR	NR
W-9	9.10	417	NR	NR
4"-Rib-1	22.40	386	NR	NR
4"-Rib-2	41.00	570	NR	NR
4"-Rib-3	33.20	455	NR	NR
8"-Rib-4	44.70	630	NR	NR
8"-Rib-5	84.20	500	NR	NR
8"-Rib-6	72.60	453	NR	NR
4"-Rib-7	28.20	532	NR	NR
4"-Rib-8	24.00	337	NR	NR
8"-Rib-9	30.00	375	NR	NR
8"-Rib-10	36.90	470	NR	NR
V-Beam-11	22.24	529	NR	NR
V-Beam-12	22.30	340	NR	NR
V-Beam-13	14.83	461	NR	NR
V-Beam-14	12.90	267	NR	NR
1[a]	57.50	378	NR	1
2[a]	120.50	565	NR	1
3[a,b]	-----	550	NR	1
4[a]	78.20	550	NR	1
5[a]	36.40	370	NR	1
6[a,b]	-----	350	NR	1

NOTES:

FAILURE MODE 1: First, slip along the seam. Second, loss of shear transfer across the sidelap, and finally, local buckling.

[a] data obtained from Reference 305

[b] tests 3 and 6 were previously subjected to several nonreversed cyclic loads at low force level, and then loaded from zero to failure under monotonic load.

NR = not reported.

Discussion of data generation is found in Section 3.1.1 of this document.

8.1.2 Data obtained from Reference 299

(Steel-Deck on Timber Framing)

TABLE A3. STEEL-DECK ON TIMBER FRAMING TEST DATA.

TEST #	SHEAR STIFFNESS lb/in.	SHEAR STRENGTH lb.	DISPL. @ PEAK IN.	FAILURE MODE
1	----	1600	NR	1
2	----	----	NR	-
3	2440	----	NR	-
4	3490	2750	NR	1
5	4180	2800	NR	1
6	2180	2650	NR	1
7	2040	2150	NR	1
8	2470	2600	NR	1
9	3150	2500	NR	1
10	3340	3200	NR	1
11	4660	3700	NR	1
12	3660	2500	NR	1
13	2060	----	NR	-
14	4200	3400	NR	1
15	5640	4200	NR	1
16	3630	2300	NR	1
17	2880	2150	NR	1
18	6210	4700	NR	1
19	4910	4600	NR	1
20	7561	7200	NR	2
21	5420	4000	NR	3
22	4420	2900	NR	4

NOTES:

Shear Stiffness values are for 40% of the strength of test A
Shear Strength values are for test B
NR : not reported.
Discussion of data generation is found in Section 3.1.2 of this document.

FAILURE MODES:

1 Splitting of the 2x4 around the screws near the seams, tearing of the sheeting around the screws near the seams and crushing of the wood around the screws near the seams.
2 The first row of screws between supports C and D pulled out of the wood purlins.
3 The chords split at the supports at corners C and D.
4 The purlins 6 in. from the support at corner C broke in tension, this purlin broke at a knot.

8.1.3 Data obtained from Reference 3

TABLE A4. STEEL-DECK TEST DATA

TEST #	SHEAR STIFFNESS K/in.	SHEAR STRENGTH Kips	DISPL. @ PEAK in.	FAILURE MODE
Q	19.50	57.40	NR	NR
R	29.50	106.00	NR	NR

NOTES:

NR = not reported.

Discussion of data generation is found in Section 3.4.1 of this document.

8.1.4 Data obtained from Reference 320.

TABLE A5. STEEL-DECK ON TIMBER FRAMING TEST DATA.

TEST #	SHEAR STIFFNESS N/mm.	SHEAR STRENGTH N/m	DISPL. @ PEAK in.	FAILURE MODE
2-A	902	2407	NR	1
3-A	967	2384	NR	1
4-A	931	2311	NR	1
2-B	789	4529	NR	2
3-B	837	4529	NR	3
4-B	732	4529	NR	2
6	852	4767	NR	2
7	958	4529	NR	2
8	967	4767	NR	3

NOTES:

NR = not reported.

Panels 2-A to 4-B were tested as a cantilever beam, panels 6-8 were tested as a simple beam.

FAILURE MODES:

1 Purlin 2x4 pulled across 2x8 at the corner C.

2 The screws in the first rib between corners C and D pulled out of the purlins.

3 The screws in the first rib between corners A and B pulled out of the purlins.

8.2 Wood Diaphragms

8.2.1 Data obtained from Reference 518

TABLE A6. WOOD DIAPHRAGM TEST DATA

TEST #	INITIAL STIFFNESS K/in.	SHEAR STRENGTH Kips	DISPL. @ PEAK in.	FAILURE MODE
1	153	44	1.30	NR
2	101	40	1.09	NR
3	88	36	1.14	NR
4	118	40	1.90	NR
5	155	48	1.95	NR
6	60	48	1.74	NR
7	29	26	1.76	NR
8	112	36	1.99	NR

NOTES:

NR = not reported.

The test data was estimated from graphs reported on Reference 518.

Discussion of data generation is found in Section 3.2.1 of this document.

8.2.2 Data obtained from Reference 3.

TABLE A7. WOOD DIAPHRAGM TEST DATA.

TEST #	INITIAL STIFFNESS K/in.	SHEAR STRENGTH Kips	DISPL. @ PEAK in.	FAILURE MODE
E	13.3	6.56	NR	NR
H	23.0	14.20	NR	NR
I	47.2	15.90	NR	NR
K	61.3	40.80	NR	NR
B	14.2	7.91	NR	NR
D	17.4	10.80	NR	NR
C	----	-----	NR	NR
N	26.2	31.90	NR	NR
P	49.7	57.80	NR	NR

NOTES:

NR = not reported.

Discussion of data generation is found in Section 3.4.1 of this document.

8.3 Steel-Deck-Reinforced Concrete Diaphragms

8.3.1 Data obtained from Reference 378.

TABLE A8. SDRC TEST DATA.

TEST No.	k_i [(1)] K/in	V_u [(2)] kips	e_p [(3)] in.	m [(4)] K/in	e_m [(5)] in.	FAILURE MODE
1	1800	---	---	-----	----	-
2	2000	136	0.1	-73.4	1.88	1
3	1600	78	0.1	-27.2	2.75	2
4	1300	70	0.1	-28.5	2.44	2
5	1700	82	0.1	-76.1	1.16	1
6	2600	97	0.4	-39.8	2.84	2
7	1500	113	0.2	-93.5	1.45	2
8	1100	42	0.1	-26.0	1.75	3
9	1900	172	0.2	-164.2	1.16	1
10	1700	131	0.4	-54.1	2.60	1
11	1600	73	0.4	-29.1	2.77	4
12	1800	131	0.2	-88.4	1.87	1
13	1900	205	0.2	-601.0	0.54	1
14	1900	172	0.4	-151.0	1.54	4
15	1300	71	0.4	-64.0	1.51	4
16	1300	75	0.4	-53.5	1.80	1
17	2200	119	0.4	-99.9	1.60	5
18	1700	120	0.2	-44.6	2.86	1
19	1300	118	0.4	-87.7	1.75	1
20	1300	76	0.2	-76.9	1.17	6
21	1200	94	0.4	-103.6	1.31	6
22	2100	139	0.4	-77.1	2.20	1
23	1700	84	0.4	-99.7	1.24	6
24	2100	124	0.4	-69.9	2.18	1
25	1900	133	0.2	-102.0	1.52	1
26	1700	64	0.1	-39.4	1.67	1
27	2000	70	0.1	-121.7	0.68	7
28	2000	93	0.2	-235.5	0.59	7
29	2300	101	0.1	-142.9	0.78	1
30	1900	93	0.1	-63.3	1.41	7
31	1500	47	0.1	-46.5	1.00	8
32	1000	39	0.1	-76.8	0.56	

[1] Initial Stiffness (k_i) based on 0.025 in. of virgin data.
[2] Average Peak Stabilized Strength (V_u).
[3] Average Displacement at Peak Strength (e_p).
[4] Degrading Slope after Peak (m).
[5] Maximum Displacement (e_m)

NOTES:

Maximum displacement and degrading slope after peak were determined by linear regression analysis on stabilized data.

Discussion of data generation is found in Section 3.3.1 of this document.

FAILURE MODES:

1 Diagonal tension.

2 Interfacial Shear.

3 Diagonal tension/shear connector.

4 Shear transfer mechanism-transverse.

5 Shear transfer mechanism-parallel.

6 Shear transfer mechanism.

7 Concrete shear failure around studs.

8 Weld failure.

8.4 Plank Diaphragms

8.4.1 Data obtained from Reference 376.

TABLE A9. PLANK DIAPHRAGM TEST DATA.

TEST No.	k_i (1) K/in	V_u (2) Kips	e_p (3) in.	m (4) K/in	e_m (5) in.	FAILURE MODE
4	1281	72.1	0.15	-11.66	5.74	1
5	2005	83.4	0.30	-6.69	9.29	1
6	1376	58.1	0.30	-6.23	8.46	2
7	1647	56.7	0.10	-5.12	8.79	2
8	716	25.6	0.30	-3.90	4.81	2
8b	1003	35.7	0.08	-2.86	10.50	2
9	1486	64.8	0.20	-8.24	6.31	1
10	2734	76.4	0.20	-15.69	4.68	2
11	2143	99.8	0.30	-45.24	2.10	3
12	1596	113.1	0.20	-33.21	3.33	3
13	2698	237.6	0.29	-55.85	2.95	3
14	3289	258.4	0.30	-40.87	3.56	3
15	2518	156.6	0.10	-56.76	2.98	1

(1) Initial Stiffness (k_i) based on virgin data.
(2) Average Peak Stabilized Strength (V_u).
(3) Average Displacement at Peak Stabilized Strength (e_p).
(4) Degrading Slope after Peak (m).
(5) Maximum Displacement (e_{max}).

NOTE:

Degrading slope after peak and maximum displacement were obtained by linear regression analysis on stabilized data.

FAILURE MODES:

1 Shear Bond

2 Tension Bond

3 Diagonal Tension

9. APPENDIX B: DIAPHRAGM SYSTEMS

Masonry buildings are constructed with various types of floor and roof diaphragm systems. Floor systems can be made of composite steel deck, timber, precast concrete, reinforced concrete and cold-formed steel.

Composite steel deck floor systems are constructed by fastening sections of cold-formed steel deck to framing members which are typically steel. The fastening may be done with arc spot welds, screws, powder driven pins, positive shear transfer devices such as headed shear studs or some combination of fasteners. Seams between adjacent panels may either be welded, screwed or button punched/crimped. A layer of concrete is placed on the deck with shrinkage and temperature steel added. Also supplemental reinforcing steel may be added in some cases, either in the negative or positive moment regions of the slab. The shear transfer device typically consists of embossments rolled into the deck profile, transverse wires attached to the deck or keystone-shaped profiles. Figure B1 illustrates some steel deck profiles.

Timber panels are constructed by fastening the sheathing material to the framing support. The sheathing material can be lumber (boards), plywood, and particle boards. Lumber sheathing is used in diagonal, double diagonal or straight patterns, and it is generally one-inch nominal thickness. Plywood is generally made up of an odd number of layers of veneer at right angles and is referred to as three-ply, five-ply, etc. Plywood is also constructed with the grain direction of adjacent plies parallel, and is often referred to as four-ply, six-ply, etc. Common thicknesses ranges from 5/16 to 1-1/8 inches. The framing support is generally made of wood members such as 2-inch dimension lumber, glulam (glued laminated timber) beams, built-up plywood beams, and open-web trusses; however, there are some systems that utilize metal framing. The connection between the sheathing material and the framing elements is made by mechanical fasteners such as nails and staples, or adhesives or a combination of both [322]. Figure B2 illustrate some wood panel types.

Precast concrete units are usually produced as standard precast members or precast-prestressed concrete members. Standard precast members utilize ordinary reinforcing bars and they are designed by standard reinforced-concrete theory. Precast-prestressed concrete members utilize stretched wires which induce a prestressing force into the element resulting in a fully or partially prestressed member. Precast members may be used as wall panels and as roof and floor elements. Wall panels are used as bearing walls or as curtain walls attached to columns and beams. Roof and floor elements are made in a variety of shapes, and in a wide range of span lengths and depths. The common shapes are flab slab, hollow-core plank, single and double tee. Figure B3 shows

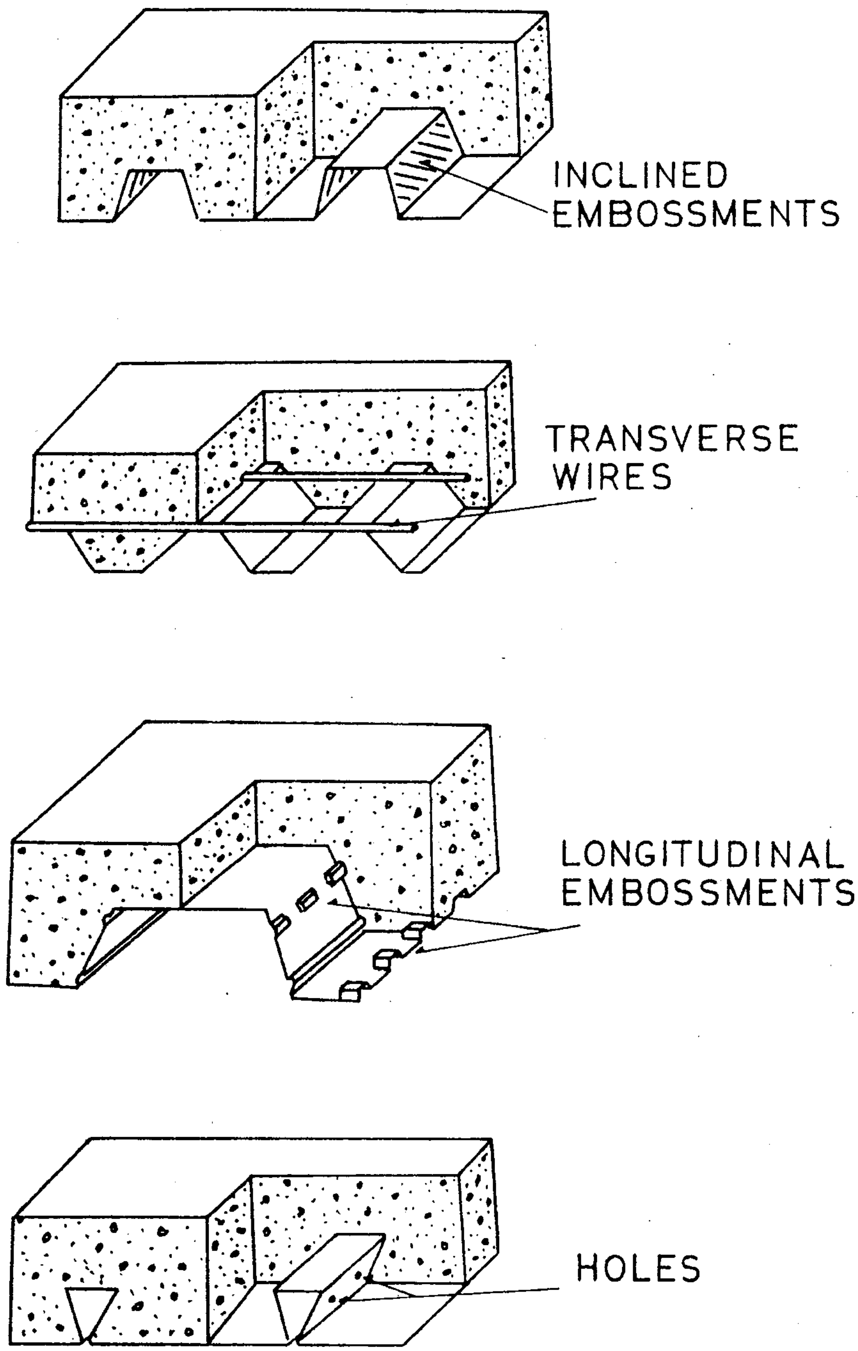

Figure B1. Typical steel-deck profiles [378]

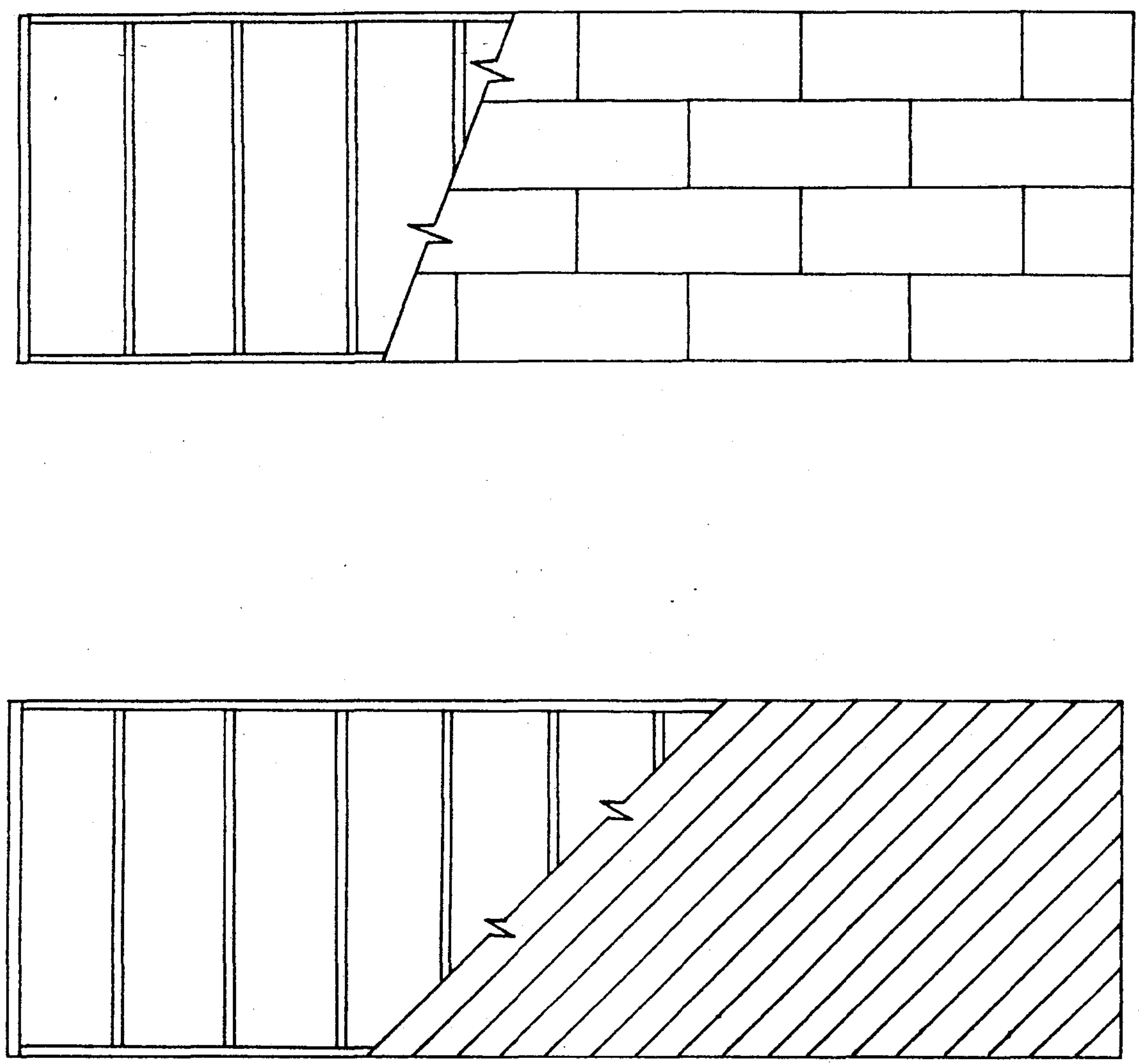

Figure B2. Plywood and diagonally sheathed panels [22,149]

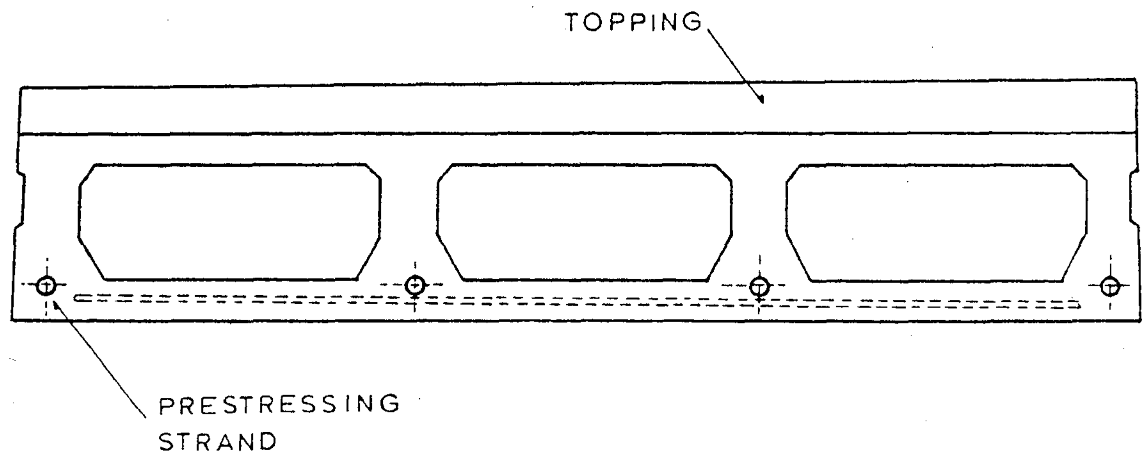

Figure B3. Typical Span-Deck hollow-core precast prestressed cross-section with topping [481] the cross section of a typical hollow-core plank.

Reinforced concrete floor and roof systems may be classified as one-way or two-way systems. One-way systems have the main reinforcement running in one direction only, two-way systems have the main reinforcement running in two perpendicular directions. The most common types for one-way systems are the slab-beam-girder systems and the one-way joist floor. The slab-beam-girder systems consists of a slab supported by a series of parallel beams supported at their extremities by girders, which in turn frame into columns. A one-way joist floor consists of a series of small closely spaced joists, framing into girders, which are in turn carried by columns. Common types for two-way systems are the two-way slab on beam, flat slab, and flat plate. Two-way slabs on beams aresolid slabs supported by beams on the column lines on each side of slab panel. Flat slabs are beamless systems with dropped panels or column capitals or both. Flat plates are beamless systems with no dropped panels or column capitals. Figure B4 illustrates typical reinforced concrete floor systems.

Cold-formed steel floor systems are made from steel sheets with thicknesses usually varying from 0.012 to 0.075 in., and with depths usually varying from 1-1/2 to 3 in. The system is constructed by fastening sections of cold-formed steel deck to framing members which are commonly steel. Floor and roof systems are capable of resisting horizontal loads in addition to gravity loads if they are interconnected to each other and adequately fastened to the supporting frame. Figure B5 illustrates typical cross sections for cold-formed steel decks.

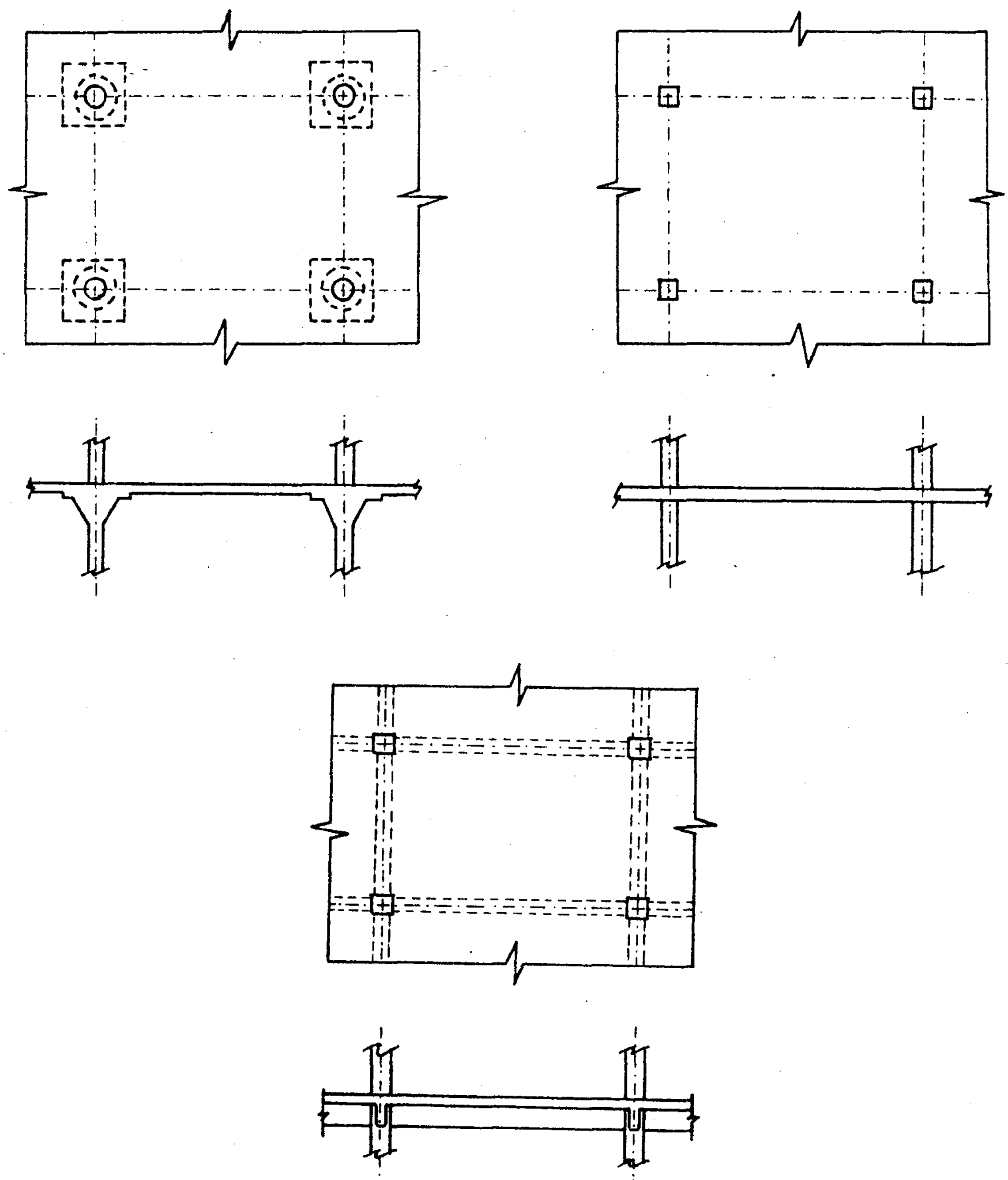

Figure B4. Typical reinforced concrete floor systems [344]

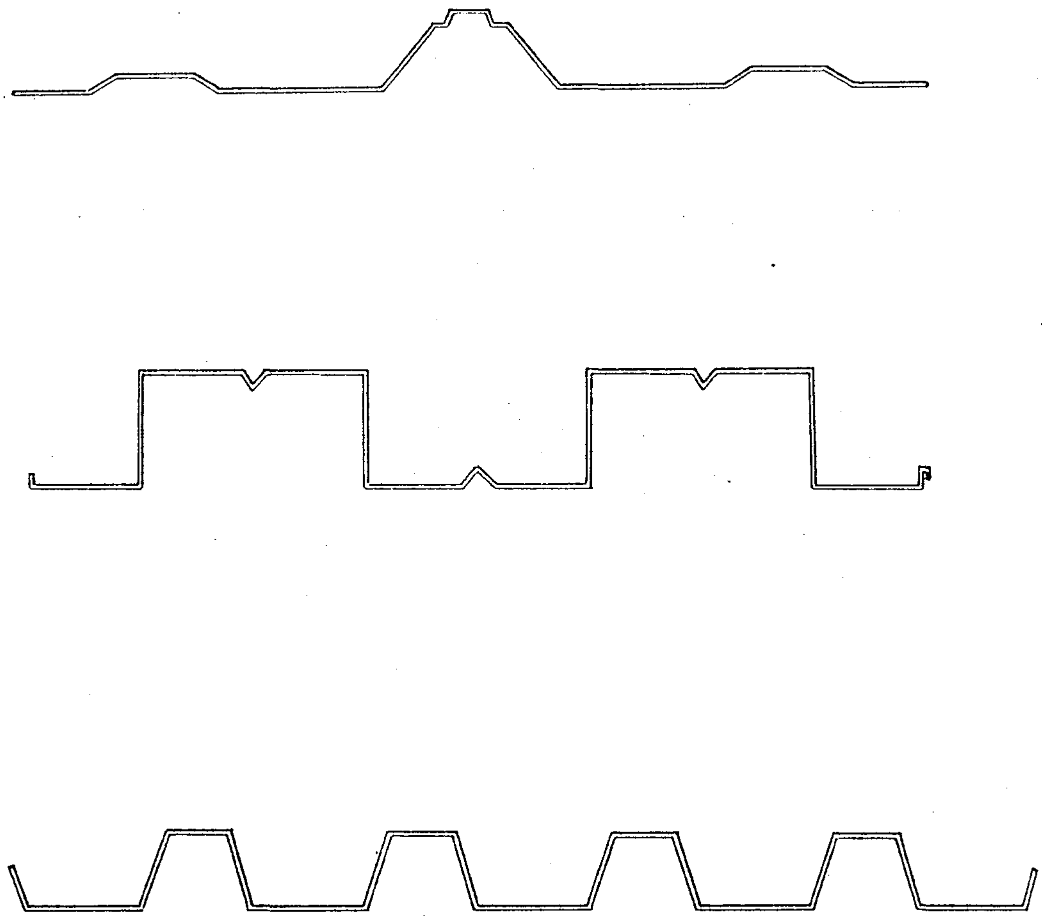

Figure B5. Typical cross-sections for Cold-Formed-Steel-Decks [113,299]