COMPARISON OF TWO-HINGED AND HINGELESS LIGHT GAGE, COLD-FORMED STEEL FRAMES

By

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INTRODUCTION

The Problem

As the American farmer's margin of profit continues to decrease, it is becoming necessary for him to find new and better ways by which he can decrease his operational costs. One method of doing this is through better and less expensive methods of constructing his farm buildings.

In the design and construction of nearly all buildings for agricultural purposes as well as light industrial buildings, the frame of the building is extended to the foundation and fastened by a connection which has little resistance to rotation. The apparent reason for this practice is that when designing a building of this type, the frame which ends at the foundation, can be assumed to be pinned. Thus the stresses can be evaluated quite easily. This also seems to be an accepted construction method which has been carried on through the years. However, through this practice a great deal of stiffness and rigidity, which could be added to the frame by extending the frame below the ground level or by using rigid fasteners, is lost. The amount of stiffness acquired by extending the frame ends below the ground level is difficult to predict, however, because of support yielding due to soil movement. This soil movement, in turn, is difficult to predict because of the large differences in soil properties due to variations in soil condition and type. Another advantantage of extending the frame ends below the ground level is that the structure can be erected without making a special foundation. Also by using this means of construction, it is possible to get a more uniform distribution of stresses in the frame. This in turn allows a frame of more nearly constant cross section to be used, thus reducing production costs.

Objectives

The objectives of this study are listed as follows:

1. Evaluate stiffness or resistance to bending of geometrically similar hingeless and two-hinged light gage, cold-formed steel frames under the same loading conditions. This is to be done experimentally and analytically to determine accuracy and reliability of analytical methods.

2. Evaluate secondary stresses developed due to movement of supports of geometrically similar hingeless and two-hinged, light gage, cold-formed steel frames. Movement of supports for two-hinged frames is to be lateral to cause separation of supports. Support movement of the hingeless frame shall be translational, rotational, or a combination which would likely simulate actual structural conditions. This is to be done experimentally and analytically.

Some of the reasons for establishing the above objectives are: (1) If a building frame is too flexible, considerable deflection might occur which could cause high stress in the covering material, causing it to fail or become damaged. (2) The effect of support movement on building frames is important because it creates secondary stresses in the frames. (3) Determine the structural advantages one frame type might

HAVE OVER THE OTHER. (4) VERIFY THEORETICAL ANALYSIS BY USE OF EXPERIMENTAL PROCEDURES.

CHAPTER II

REVIEW OF LITERATURE

Introduction

Robinson (20) stated, "Since the end of the war, changes in husbandry practice have brought about a revolution in farm buildings. In particular with increased mechanization, larger buildings and greater space for working are required. The design of the future must reduce to a minimum the interference with adaptability and free movement." This author also suggested that building methods in the future would undoubtedly make greater use of pre-fabricated units made in the factory and assembled on the farm. It seems that this theory goes along very well with the use of rigid frames for farm buildings. Curtis and Hansen (5) stated that lumber rigid frames may revolutionize the construction of farm buildings. The reasons they gave were that rigid frames provide more usable space; are cheaper in most cases; and they could be erected quickly and easily.

Paul and Hansen (17) investigated the possibility of using concrete rigid frames for buildings free of any interior supports, trusses, or braces. The objective of this study was to develop a series of designs for use by concrete precasting plants, where the frames would be cast and then shipped to the site to be constructed. They also found that the behavior of the concrete rigid frame tested could be

PREDICTED QUITE ACCURATELY BY THEORETICAL MEANS.

EFFECTS OF SUPPORTS PLACED IN THE SOIL

ANALYTICAL EVALUATION OF SUPPORTS

IN THE DESIGN OF LIGHT STRUCTURAL FRAMES WITH FIXED-END SUPPORTS, DEPENDING UPON THE SOIL FOR RESISTANCE TO MOVEMENT, THE PROBLEM ARISES AS TO HOW MUCH RESISTANCE CAN BE EXPECTED FROM THE SOIL. THE MAIN REASONS FOR THIS PROBLEM ARE THE WIDE VARIATION OF MECHANICAL PROPERTIES OF THE SOIL, AND THE INABILITY TO PREDICT STRESSES IN SOIL SUBJECTED TO LATERAL PRESSURES FROM A PILLAR.

IF THE ENDS OF THE FRAMES ARE PLACED IN A HOLE DUG IN THE GROUND AND THE VOIDS FILLED WITH CONCRETE OR OTHER SUITABLE MATERIAL, THEN IT APPEARS THAT THE END CONDITIONS COULD BE CONSIDERED TO BE THE SAME AS PILES OR POLES PLACED IN THE GROUND. FOR THIS REASON THE BEHAVIOR OF LATERALLY LOADED PILES WAS INVESTIGATED.

A RATIONAL SOLUTION WAS PROPOSED BY CZERNIAK (6) FOR THE RESISTANCE TO OVERTURNING OF SHORT PILES IN WHICH HE ASSUMED THE PILE TO BE PERFECTLY RIGID AND THE SOIL RESISTANCE INCREASED LINEARLY WITH DEPTH. THE SOIL RESISTANCE WAS BASED ON THE FOLLOWING THEORY: WHEN A SHORT PILE--A PILE WITH EMBEDDED DEPTH NOT OVER TEN TIMES ITS LEAST LATERAL DIMENSION--IS ROTATED IN ITS POSITION, THE HORIZONTAL PRESSURE AGAINST THE PILE INCREASES UNTIL IT REACHES THE LIMITING VALUE KNOWN AS THE PASSIVE EARTH PRESSURE. FURTHER DISPLACEMENT OF THE PILE DOES NOT SIGNIFICANTLY CHANGE THE PRESSURE. BEFORE THE PASSIVE PRESSURE IS REACHED, THE BODY OF THE EARTH IS IN A STATE OF ELASTIC EQUILIBRIUM AND THE MAGNITUDE OF THE PRESSURE IS RELATED TO THE AMOUNT OF PILE MOVEMENT. THE MOVEMENT AT THE GROUND LEVEL REQUIRED TO DEVELOP THIS PASSIVE PRESSURE MAY BE AS HIGH AS 1/32 INCH PER FOOT OF PILE EMBEDMENT. THE GENERAL FORMULA GIVEN

TO ESTIMATE PASSIVE PRESSURE IS:

$$P_H = K \rho H \tan^2 \left(45^\circ + \frac{\phi}{2}\right) + 2 c \tan \left(45^\circ + \frac{\phi}{2}\right)$$

ρ = UNIT WEIGHT OF SOIL, LB/FT^3

H = DEPTH, FT

c = COHESION, LB/FT^2

ϕ = ANGLE OF FRICTION IN DEGREES

P_H = HORIZONTAL PRESSURE, LB/FT^2

K = EFFICIENCY FACTOR TO ALLOW FOR ROUGHNESS OF PILE FOR ROUND BORED FOUNDATIONS, APPROXIMATELY 2

USING THIS THEORY AS A BASIS, HE THEN DEVELOPED EQUATIONS TO DETERMINE ACTUAL SOIL PRESSURES AND REQUIRED PILE EMBEDMENT FOR ROUND AND RECTANGULAR SECTIONS. CZERNIAK'S EQUATIONS FOR ACTUAL SOIL PRESSURES WERE:

(A) ROUND SECTION

$$P_X = 9.425 \frac{H_o}{L} - \left[4E + 3\right] \frac{X}{L} + 2 \left[3 \frac{E}{L} + 2\right]\left[\frac{X}{L}\right]^2$$

(B) RECTANGULAR SECTION

$$P_X = 6 \frac{H_o}{L} - \left[4 \frac{E}{L} + 3\right]\frac{X}{L} + 2 \left[3 \frac{E}{L} + 2\right]\left[\frac{X}{L}\right]^2$$

P_X = EARTH PRESSURE AGAINST PILE AT DISTANCE X FROM RESISTING SURFACE, LB/FT^2

H_o = LATERAL FORCE PER FOOT OF PILE DIAMETER APPLIED AT THE RESISTING SURFACE, LB/FT

E = DISTANCE FROM LATERAL LOAD TO RESISTING SURFACE, FT

L = DEPTH OF PILE, MEASURED FROM THE RESISTING SURFACE, FT

X = DISTANCE BETWEEN POINT AT WHICH P_X IS TAKEN AND RESISTING SURFACE

USING THESE EQUATIONS AND THE RECOMMENDED LATERAL SOIL PRESSURES

TABLE I

RECOMMENDED LATERAL SOIL PRESSURE
IN POUND PER SQ FT PER FOOT DEPTH

Class of Material	Value
Rock in natural bed-limited by the stress in pile	
Medium hard caliche	500
Fine caliche with sand layers	400
Compact well graded gravel	400
Hard dense clay	400
Compact coarse sand	350
Compact coarse and fine sand	300
Medium stiff clay	300
Compact fine sand	250
Ordinary silt	200
Sandy clay	200
Adobe	200
Compact inorganic sand and silt mixtures	200
Soft clay	100
Loose organic sand and silt mixtures and mulch or bay mud	0

in Table I the design for a pile can be made. The accuracy of solutions made in this manner is certainly questionable for piles placed in the upper five or six feet of the earth as would be the case in farm buildings. It does, however, provide an estimate that could be used.

Matlock and Reese (12) proposed rational solutions in which the non-linear force-deformation characteristics of the soil were considered. The basic equation used for the elastic-pile theory was:

$$\frac{d^4Y}{dX^4} + \frac{E_s}{EI} Y = 0$$

Y = lateral deflection in inches

X = depth below ground line in inches

E_s = soil modulus lb/in^2

EI = flexural stiffness of pile lb-in^2

and for the rigid pile theory

$$p = -E_s Y_T - E_s SX$$

S = slope of the pile

Y_T = deflection at $X = 0$

p = soil reaction per unit of length of pile lb/in

Then using dimensional analysis, non-dimensional parameters were obtained and substituted into the above equations. By using different soil modulus constants in the equation $E_s = KX^n$, where

K = constant of soil modulus

n = exponent

and by using repeated applications of the preceding theories, a satisfactory solution can then be obtained in the structure-pile-soil system.

The latter of the two solutions would appear to be more appropriate since soil modulus varies not only with depth, but also with width of the pole, the magnitude of the applied load, and the deflection. However, an accurate prediction of the soil modulus variation $E_X = KX$ at relatively shallow depths, would be hard to achieve. This would also vary considerably for each location a building was to be constructed.

Nelson (14) derived an equation for the deflection of an elastic pole under lateral load when the deflection due to anchorage yield was known. The assumptions used in his derivation were:

(1) Pole was loaded by tilting moments in vertical plane and rotates about a fixed point

(2) Concrete used for backfilling the anchorage did not contribute to the stiffness of the pole

(3) Horizontal reaction on the pole during application of tilting moments is distributed as a parabola with horizontal axis

Using these assumptions and the equation

$$P = EI \frac{d^4Y}{dX^4}$$

P = below-grade reaction on the pole

EI = flexural stiffness of pole

Y = lateral deflection

X = depth below ground line

the equation for the slope (ϕ) of the pole axis at the ground line was developed. This equation is:

$$\phi = \frac{dY}{dX} = -\frac{\delta}{d} - \frac{\alpha^2 P D^2}{2\,EI}\left[\frac{\alpha^3}{15\quad(2-3\alpha)} + \frac{1}{\gamma\alpha} + \frac{1}{3}\right]$$

$$\alpha = \frac{d}{D} \qquad\qquad \gamma = \frac{D}{H}$$

δ = HORIZONTAL DEFLECTION AT GROUND LINE

d = DISTANCE BETWEEN GROUND LINE AND POINT OF ROTATION

D = TOTAL DEPTH OF SET OF THE POLE

H = DISTANCE BETWEEN GROUND LINE AND POINT OF APPLIED LOAD

P = LATERAL LOAD

BY USING 0.6 AS A VALUE FOR $\frac{d}{D}$, WHICH SEEMED TO BE APPROPRIATE FROM TEST RESULTS OF VARIOUS OTHER EXPERIMENTERS, HE OBTAINED REASONABLE RESULTS WHEN COMPARED TO TEST RESULTS OBTAINED BY NELSON AND HIS ASSOCIATES (15). THESE TESTS WERE, HOWEVER, FOR ONLY ONE MAJOR TYPE OF SOIL CONDITION. THE MAJOR OBJECTION TO THIS METHOD IS THAT DEFLECTION AT THE GROUND LINE HAS TO BE MEASURED OR ESTIMATED.

PROTOTYPE SUPPORT STUDIES

A STUDY OF THE RESISTANCE TO OVERTURNING OF UTILITY POLES WAS MADE BY ANDERSON (1). HE FOUND THAT THE FAVORED DESIGNS FOR RESISTING TILTING MOMENTS WERE SLIM AND DEEP WITH THE TOP THIRD OF THE FOUNDATION INCREASED IN WIDTH AT RIGHT ANGLES TO DIRECTION OF FORCE. THE THEORY WAS USED THAT UNIT DEFLECTION VARIED WITH DEPTH AND WITH CERTAIN CHARACTERISTICS OF THE SOIL, WHICH WERE ASSUMED TO VARY LINEARLY WITH DEPTH. THE NET RESISTANCE OF THE SOIL TO HORIZONTAL MOVEMENT WAS CONSIDERED TO BE THE DIFFERENCE OF THE PRESSURES ON ITS TWO SIDES OR PASSIVE RESISTANCE LESS ACTIVE PRESSURE. THEN USING GENERAL EQUATIONS AND TAKING MOMENTS ABOUT AN ASSUMED NEUTRAL AXIS, IT WAS FOUND THAT THE ERRORS WERE ON THE SAFE SIDE AND ACCURACIES WERE ABOUT 5 PER CENT ASSUMING SOIL VALUES TO BE ABSOLUTE.

MCCELLAND AND FOCHT (13) CONDUCTED AN EXPERIMENT ON A 24 INCH

PILE DRIVEN INTO THE GROUND 75 FEET. FROM THESE EXPERIMENTS THEY FOUND THE SOIL MODULUS TO VARY ALMOST LINEARLY WITH DEPTH IN WHICH THE SOIL WAS A CONSOLIDATED CLAY. THEY ALSO FOUND THAT SOIL MODULUS VARIED WIDELY WITH DEPTH AND PILE DEFLECTION.

THE PREVIOUSLY MENTIONED EXPERIMENTS WERE CARRIED OUT WITH MUCH LARGER POLES AND DEEPER SETTINGS THAN WOULD BE EXPECTED IN FARM CONSTRUCTION. THEREFORE, MOST OF THIS DESIGN DATA WOULD NOT APPLY TO POLE FOOTING DESIGNS OF AGRICULTURAL BUILDINGS SINCE THESE ARE RELATIVELY SHALLOW.

A STUDY OF THE LATERAL LOAD EFFECTS ON POLES WAS MADE BY NELSON (15) AND ASSOCIATES ON SIX INCH DIAMETER POLES PROJECTED 14 FEET ABOVE GROUND SURFACE. IT WAS FOUND THAT THE DEPTH OF SETTING WAS ONE OF THE IMPORTANT FACTORS IN STABILITY OF THE POLE ANCHORAGES. BY INCREASING DEPTH 3 1/2 AND 5 FEET IT WAS FOUND THAT MOVEMENT WAS REDUCED 38 PER CENT AND 30 PER CENT RESPECTIVELY OF THE VALUE AT 2 1/2 FEET. THIS EFFECT WAS FOUND TO BE MOST PRONOUNCED ON DEFLECTION RATES DURING THE FIRST APPLICATION OF LOADS. OTHER IMPORTANT FINDINGS WERE THAT RELATIVELY SMALL INCREASES IN WATER CONTENT OF THE SOIL CAN CAUSE RADICAL LOSS OF STABILITY IN CLAY SOILS, AND THAT ROTATION OCCURED ABOUT WELL-DEFINED POINTS WHICH VARIED IN DEPTH FROM 1/2 TO 2/3 OF THE TOTAL DEPTH. THE FOLLOWING METHODS WERE SUGGESTED TO REDUCE POLE ROTATION:

(1) THE USE OF CONCRETE AS COMPARED TO TAMPED EARTH FOR BACKFILLING AROUND THE PILE

(2) INCREASING THE DEPTH OF THE POLE SETTING

(3) KEEPING SOIL AROUND THE ANCHORAGE DRY

(4) PRECONSOLIDATION OF SOIL AROUND THE ANCHORAGE TO INCREASE SOIL ELASTIC MODULUS

From these same tests Nelson (14) found the following: that by a combination of preconsolidation, the use of complete concrete encasement, and a 5 foot anchorage depth, total rotation of a pole at the ground line can be as low as approximately 3×10^{-6} radians per ft-lb of applied tilting moment. This was for a nominal 5 inch top pressure creosoted southern pine pole in a 12-inch diameter concrete anchorage. It was also found that approximately 3 per cent of this rotation was non-recoverable because of plastic consolidation of soil, and that approximately 50 per cent was caused by elastic consolidation.

Model Support Studies

Various experimenters have used model tests to study lateral loading on piles. However, most of the studies were not carried out in such a manner that physical similarity could be obtained.

Beckett (3) made a study of laterally loaded model poles using principles of similitude. In these tests used to predict the behavior of laterally loaded poles, it was found that the deflection of the prototype was close to that predicted by the model in all cases. These tests were run in three different soil types: loose sand, dense sand, and saturated sandy clay. From these tests the following prediction equations were made.

For loose sand

$$y = 1.824 \times 10^{-5} D (P/D\gamma)^{12.5(D/H)^{0.68}}$$

For dense sand

$$y = 1.68 \times 10^{-3} D\, e^{5.5/(D^{0.87} H^{2.13} \gamma)}$$

For saturated sandy clay

$$y = 632\, D(H^3\gamma)^{3.2546} (kt/D)^{0.8009}$$

Y = LATERAL MOVEMENT

D = POLE DIAMETER

P = APPLIED LOAD

H = DEPTH OF EMBEDMENT

γ = WEIGHT OF SOIL PER UNIT VOLUME

K = COEFFICIENT OF PERMEABILITY

T = TIME ELAPSED SINCE LOADING

THESE EQUATIONS ARE DIRECTLY APPLICABLE TO ANY SIZE OF POLE PROVIDED THEY MEET THE REQUIREMENTS OF DIMENSIONAL ANALYSIS USED IN THE TESTS. THIS WOULD MAKE THESE EQUATIONS GOOD ONLY FOR THE THREE SOIL TYPES TESTED. IT WAS ALSO FOUND THAT $P/D^3\gamma$ VERSUS THE LOAGRITHM OF Y/D PLOTTED ON RECTANGULAR COORDINATE PAPER RESULTED IN A STRAIGHT LINE.

RICE (19) CONDUCTED MODEL EXPERIMENTS TO MEASURE RIGIDITY OF SELECTED ANCHORAGE DESIGNS UNDER APPLIED BENDING AND HORIZONTAL LOADS. THESE TESTS WERE DESIGNED AND OPERATED ACCORDING TO PRINCIPLES OF SIMILITUDE AND CONDUCTED IN A SAND TANK FILLED WITH DENSE SAND. THE BASIS OF SELECTION FOR THE ANCHORAGES WAS THAT A HORIZONTAL EXTENSION ATTACHED TO THE FIXED-END ANCHORAGE BELOW THE GROUND LEVEL WOULD REDUCE SOIL PRESSURE AND THEREBY INCREASE THE RIGIDITY OF THE ANCHORAGE. EXPERIMENTS WERE CONDUCTED USING HORIZONTAL EXTENSIONS OR WINGS WITH WING LENGTH TO ANCHORAGE DIAMETER RATIOS OF 2 AND 3. WING DEPTH TO ANCHORAGE DEPTH RATIOS OF 1/3 AND 1/2 WERE USED WITH THE WINGS LOCATED AT THE UPPER PORTION OF THE ANCHORAGES WHERE SOIL PRESSURES ARE GREATEST UNDER OVERTURNING LOADS. FOR A TYPICAL VALUE OF 20 FOR $P/\gamma D^3$,

P = APPLIED LOAD

γ = SOIL DENSITY

D = ANCHORAGE DIAMETER

IT WAS FOUND THAT THE MOVEMENT WAS REDUCED APPROXIMATELY 20 PER CENT

BY A WING TWICE AS WIDE AS THE ANCHORAGE DIAMETER AND APPROXIMATELY 40 PER CENT BY A WING 3 TIMES THE ANCHORAGE DIAMETER AS COMPARED TO AN ANCHORAGE WITHOUT A STABILIZING WING NORMAL TO THE MOMENT PLANE. IT WAS ALSO FOUND THAT A WING IN A PLANE PARALLEL TO THE PLANE OF APPLIED MOMENT WAS MORE EFFECTIVE THAN A NORMAL WING FOR ANCHORAGES WITH A DEPTH TO DIAMETER RATIO OF 4, WHILE FOR RATIOS OF 7 AND 9 A NORMAL WING WAS FOUND TO BE MORE EFFECTIVE.

FIXED-END SUPPORTS

TO DATE THERE IS VERY LITTLE INFORMATION PUBLISHED ON THE EFFECT OF FIXED-END SUPPORTS IN LIGHT STRUCTURES.

SALMON (21) IN AN ARTICLE ON MOMENT-ROTATION CHARACTERISTICS OF COLUMN ANCHORAGES STATED THAT THE THREE TYPES OF LIKELY FAILURE OF COLUMN ANCHORAGES ARE, (1) FAILURE IN SHEAR RESISTANCE, (2) FAILURE IN MOMENT RESISTANCE, AND (3) FAILURE IN TENSILE RESISTANCE. HE ALSO STATED THAT SHEAR FAILURE WAS MOST LIKELY TO OCCUR IN LOW, WIDE BUILDINGS. A TENSILE OR MOMENT RESISTANCE FAILURE IS MORE LIKELY TO OCCUR IN TALLER, NARROWER BUILDINGS.

GALAMBOS (7) IN A RATIONAL DERIVATION FOUND THAT THE BUCKLING STRENGTH OF A PINNED-BASE RIGID FRAME WAS CONSIDERABLY LOWER THAN THAT OF AN IDENTICAL FIXED-BASE FRAME. USING HIS DERIVATION HE FOUND THAT A FIXED-BASE FRAME COULD CARRY 4.07 TIMES AS MUCH AXIAL LOAD AS A PINNED-BASE CONDITION. HOWEVER, FURTHER RESEARCH ON THE MOMENT-ROTATION CHARACTERISTICS OF COMMON COLUMN FOUNDATIONS IS NECESSARY BEFORE MORE ACCURATE ESTIMATES OF BASE RESTRAINT CAN BE MADE. AVAILABLE INFORMATION INDICATES THAT PRESENTLY USED PINNED-COLUMN BASES GIVE ENOUGH RESTRAINT TO INCREASE THE BUCKLING STRENGTH OF THE frame considerably. In this same article he stated that, "Pinned column bases are specified in most of the United States because the construction of suitable foundations for fixed-bases usually increases the over-all cost." However, in this statement he is probably referring to larger structures than would ordinarily be used on the farm or for light industrial purposes.

The only information on actual tests of fixed-end supports in structural frames found were those conducted by Nelson (16) and his associates. These tests were conducted by the use of models and prototype structures. The type of frames tested were one-hinged arch frames and three-hinged arch frames which were for comparisons. The prototype frames were glue-laminated wooden arches. Models used were one-half size wooden arches and one-eighth size models using mild steel arches. The stiffness of model and prototype one-hinged frames with moment resisting anchorages in sand and clay soils ranged from 1.55 to 1.85 times as great as the stiffness of identical three-hinged arches under short-term loads applied at the crown. This compared very closely to the theoretical stiffness increase of 1.78 for a typical prototype with ideal fixed-end anchorages. Other findings were that stabilizing wings on prototype arch anchorages gave 52 per cent less movement of the anchorage as compared to anchorages without wings. These wings were one-third the anchorage diameter. Also the models were found to give reliable and useful information.

Frame Properties

Since the frames tested in the present study were made up of

LIGHT GAGE, COLD-FORMED STEEL SECTIONS, IT WAS THOUGHT THAT INFORMATION WAS NEEDED WITH RESPECT TO PROPERTIES OF THESE TYPE SECTIONS.

LIGHT GAGE, COLD-FORMED STEEL SECTIONS ARE MADE BY COLD FORMING SHEET OR STRIP STEEL IN ROLLS OR BRAKES. THE GENERAL RANGE OF THICKNESS IS FROM NO. 10 GAGE (0.1345 IN) TO NO. 28 GAGE (0.0149 IN). HOWEVER, OTHER THICKNESSES HAVE BEEN USED.

WINTER (23) GAVE THESE THREE REASONS FOR USING COLD-FORMED STRUCTURAL SECTIONS AS COMPARED TO HOT-ROLLED SECTIONS.

(1) WHERE MODERATE LOADS AND SPANS RENDER THE THICKER HOT-ROLLED SHAPES UNECONOMICAL

(2) WHERE, REGARDLESS OF THICKNESS, MEMBERS ARE WANTED OF CROSS-SECTIONAL CONFIGURATIONS WHICH CANNOT ECONOMICALLY BE PRODUCED BY HOT-ROLLING OR BY WELDING OF FLAT PLATES

(3) WHERE IT IS DESIRED THAT LOAD-CARRYING MEMBERS ALSO PROVIDE USEFUL SURFACES, SUCH AS IN FLOOR AND WALL PANELS, ROOF DECKS AND THE LIKE

FROM THE THIRD REASON IT IS APPARENT THAT THE COLD-FORMED SECTIONS CAN BE DIVIDED INTO TWO GENERAL CLASSES--INDIVIDUAL STRUCTURAL SECTIONS AND DECKS OR PANELS.

FROM EXPERIMENTAL RESULTS GRIFFIN (8) CONCLUDED THAT THERE ARE THREE MODES OF FAILURE IN COLD-ROLLED SECTIONS DEPENDING ON THE ACTUAL PROPORTIONS OF THE MEMBER AND THE STRUCTURAL APPLICATION. THESE MODES OF FAILURE ARE:

(1) LOCAL INSTABILITY OR PLATE BUCKLING

(2) MATERIAL FAILURE

(3) LATERAL OR OVER-ALL INSTABILITY

HE ALSO STATED THAT MOST SECTION SHAPES ARE SUBJECT TO TORSIONAL INSTABILITY DEPENDING ON THE ACTUAL FORM AND DIMENSIONS OF THE SHAPE.

IN THE DESIGN OF STRUCTURES USING COLD-FORMED STEEL SECTIONS, procedures must be used to take into account the tendency of the thin sections to buckle under compressive stresses less than the yield point of steel. A number of procedures used in the design of light gage, cold-formed steel structures are listed in the Light Gage, Cold-Formed Steel Design Manual (11). Only a few of these concerned with the shape of section used in this study will be mentioned.

Most cold-formed structural members are formed of sheet or strip steel in which the flat-width to thickness ratios of the individual components of the sections are so large that they will buckle at stresses below the yield point if subjected to compressive shearing, bending, or bearing forces. Therefore, it is necessary to design such members so that at design loads, adequate safety exists against failure by local buckling. For the shape of members to be used in this study, the local buckling would probably take place as shown in Figure 1. The compression stresses over width w would be distributed in a manner as shown in Figure 2. This non-uniformity increases with load as can be observed in Figure 2 as the load is increased from load 1 to load 2. It is difficult to account for this non-uniform stress distribution in design, so a concept called "effective design width" is used. The total compressive force over width w is equal to the area under the stress distribution curve times the thickness of the element. By use of "effective design width" the non-uniformity of the stress distribution of the actual member is replaced by one of reduced effective width b, and with constant stress of magnitude F_{MAX}. If the effective width has been chosen so that two rectangular areas, $F_{MAX} \times b/2$, shown by the dashed line in Figure 2, are equal to the area under the actual stress distribution curve, the two elements will be equivalent.

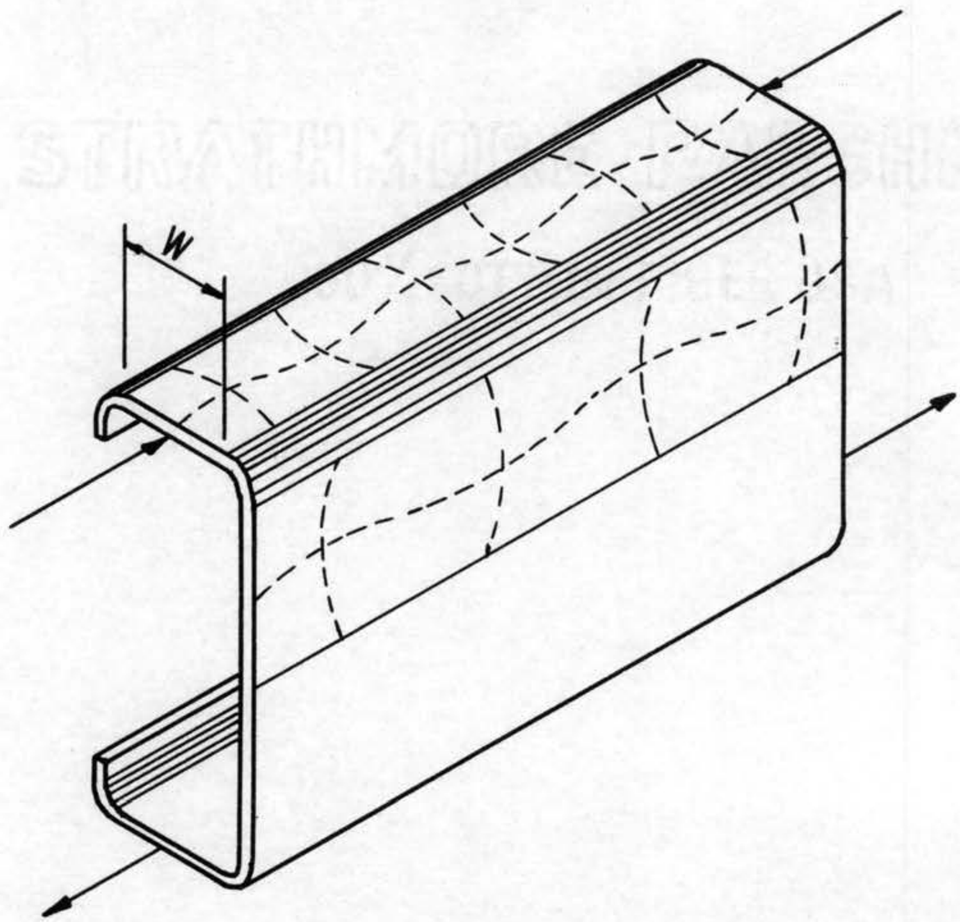


FIGURE 1. POSSIBLE LOCAL BUCKLING OF TEST SECTION

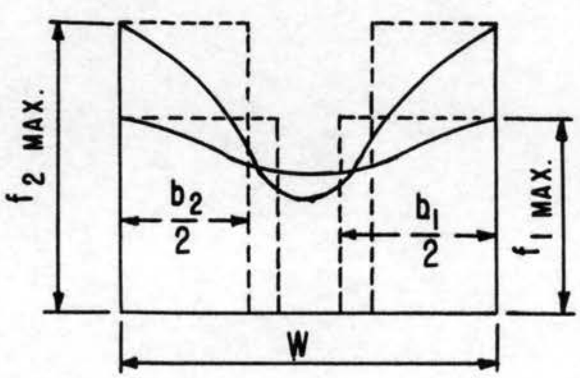


FIGURE 2. STRESS DISTRIBUTION IN BUCKLING PORTION OF SECTION.

It is also shown in Figure 2 that the effective width decreases with increasing edge stress. To determine the effective width the following equation is used (23).

$$\frac{B}{T} = 1.9\sqrt{\frac{E}{F_{MAX}}}\left[1 - \frac{0.475}{W/T}\sqrt{\frac{E}{F_{MAX}}}\right]$$

E = modulus of elasticity

T = thickness of section

B = effective width

Charts are also available for determining the effective width in the Light Gage, Cold-Formed Steel Design Manual (11).

If the effective area of the compression flange of a beam decreases as the load increases, the neutral axis will tend to move toward the tension flange. This in turn modifies the effective cross section properties such as area, moment of inertia, and the section modulus. In design work these changing properties must be accounted for. For this reason a number of the section properties in the Design Manual (11) are given for two basic stresses, F_B = 20,000 psi and 30,000 psi.

Another problem of most light gage, cold-formed steel sections is the lateral deflection or twisting due to applying the loads in the plane of the web when the section lacks symmetry about that plane. This lack of symmetry about a vertical plane or the so-called shear-center of a channel is neither coincidental with the centroid nor is it located in the plane of the web. That point in the plane of a beam section through which a transverse load must act in order to produce bending without twisting is the shear-center. In a channel this point is located a distance, M, back of the mid-plane of the web as shown in Figure 3. Since the internal shear force passes through

THE CENTER OF SHEAR AND IF THE EXTERNAL LOAD IS APPLIED IN PLANE OF THE WEB, IT WILL PRODUCE A TWISTING MOMENT, Q_M. THESE TORQUES MUST BE BALANCED BY SOME EXTERNALLY APPLIED COUNTER-TORQUES OR UNDESIRABLE TWISTING WILL RESULT. TO DETERMINE THE DISTANCE, M, THE FOLLOWING EQUATION IS GIVEN (11).

$$M = \frac{W^2}{2W + \frac{H}{3}}$$

W = PROJECTION OF FLANGES FROM INSIDE FACE OF WEB IN

H = DEPTH OF CHANNEL OR BEAM INCH

THEN TO DETERMINE MAXIMUM PERMISSIBLE SPACING OF WELDS OR OTHER CONNECTORS JOINING TWO CHANNELS TO FORM AN I-SECTION FOR FLEXURAL MEMBERS IS:

$$S_{MAX} = \frac{L}{6},$$

AND IN NO CASE SHALL THE SPACING EXCEED

$$S_{LIM} = \frac{2c\ S_W}{M\ Q}$$

S_{LIM} = MAXIMUM SPACING BETWEEN CONNECTIONS

L = SPAN OF BEAM, IN

S_W = STRENGTH OF CONNECTION IN TENSION, LB

C = VERTICAL DISTANCE BETWEEN THE TWO ROWS OF CONNECTIONS NEAR OR AT THE TOP AND BOTTOM FLANGES, IN

Q = INTENSITY OF LOAD ON BEAM, LB/LINEAR IN

M = DISTANCE OF SHEAR CENTER FROM MID-PLANE OF THE WEB, IN

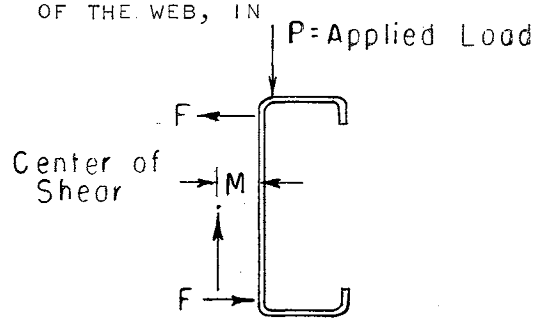


FIGURE 3. FORCES IN THE PLANE OF THE SECTION.

CHAPTER III

THEORETICAL ANALYSIS

Effect of Loading Conditions

Any structure should be designed for the maximum expected stresses. To estimate the maximum stresses, the different loading conditions which might produce these need to be investigated. In this study the following loading conditions were considered: vertical loads due to snow or ice, loads due to wind forces, and grain loads.

The methods used to calculate the bending moments and perpendicular shearing stresses due to the various loading conditions were the moment-area method (18) and formulas developed by Kleinlogel (10). Before applying these methods, however, one should consider the assumptions used in the development of these methods. These are as follows: the material behaves elastically and deformations due to shearing forces are neglected. Kleinlogel's Formulas also assumed no rotation or displacement of fixed supports, and no displacement of hinged supports occur. Keeping these assumptions in mind, the next step is to consider how the previously mentioned methods would be applied to a typical hingeless and two-hinged frame. For this purpose consider the following frame, Figure 4, with a concentrated load at the peak. This would represent an example of a hingeless frame with supports A and E fixed.

THE MOMENT-AREA METHOD IS APPLIED AS FOLLOWS:

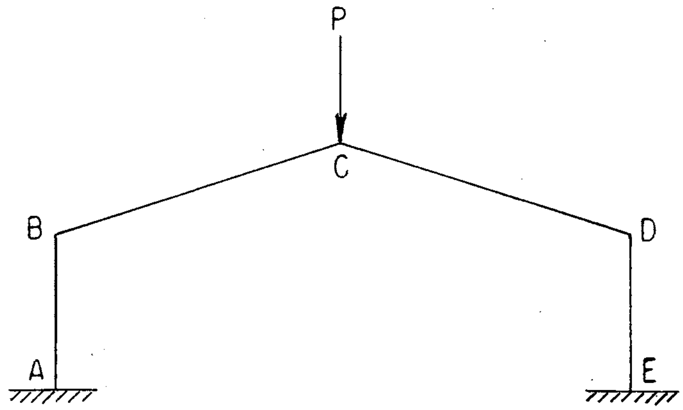


FIGURE 4. EXAMPLE OF HINGELESS FRAME.

ASSUME NO ROTATION OCCURS AT POINTS A AND C AND NO DISPLACEMENT OF C IN HORIZONTAL OR X DIRECTIONS DUE TO SYMMETRY OF FRAME AND LOADING. THEN ASSUMING POINT C FIXED, AND BY THE FREE BODY DIAGRAM OF FIGURE 5,

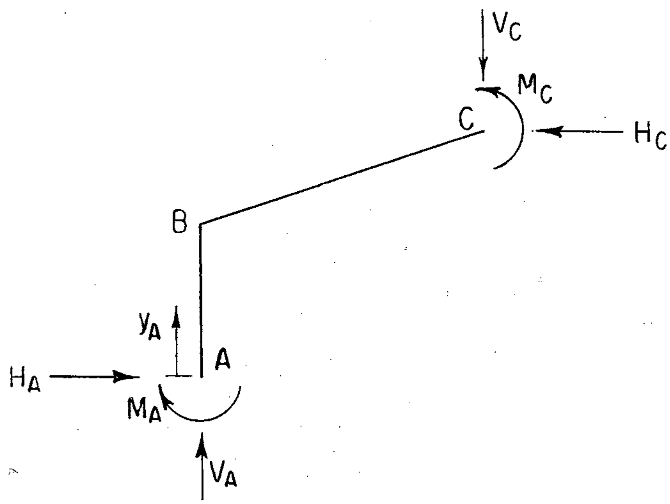


FIGURE 5. FREE BODY DIAGRAM OF FRAME MEMBER

THE FOLLOWING TWO MOMENT-AREA EQUATIONS CAN BE WRITTEN:

$$\Delta \phi = \int_C^A \frac{M\,ds}{EI} = 0 \qquad \Delta X = \int_C^A Y_A \frac{M\,ds}{EI} = 0$$

IN WHICH

$\Delta \phi$ = ANGULAR ROTATION, RADIANS, OF TANGENT TO FRAME AT C WITH RESPECT TO TANGENT OF A

ΔX = HORIZONTAL DISPLACEMENT OF A WITH RESPECT TO C

Y_A = VERTICAL DISTANCE FROM A TO POINT ON THE FRAME WHERE "M" IS APPLIED

M = BENDING MOMENT IN ELEMENT DS

DS = DIFFERENTIAL ELEMENT ALONG FRAME

E = MODULUS OF ELASTICITY

I = MOMENT OF INERTIA

THESE TWO EQUATIONS CAN BE SOLVED SIMULTANEOUSLY FOR M_A AND H_A. IF POINTS A AND E ARE ASSUMED TO BE PINNED AS IS THE CASE FOR A TWO-HINGED FRAME, WE CAN AGAIN CONSIDER THE FREE BODY DIAGRAM EXCEPT IN THIS CASE M_A WOULD EQUAL ZERO AND ONLY ONE MOMENT-AREA EQUATION WOULD NEED TO BE CONSIDERED, WHICH IS:

$$\Delta X = \int_C^A Y_A \frac{M\,DS}{EI} = 0$$

THIS EQUATION CAN THEN BE SOLVED DIRECTLY FOR H_A.

ANALYSIS BY THE USE OF KLEINLOGEL'S FORMULAS CONSISTS ONLY OF PLACING KNOWN VALUES IN GIVEN FORMULAS AND SOLVING DIRECTLY. AN EXAMPLE WAS, THEREFORE, NOT CONSIDERED NECESSARY.

THE SIGN CONVENTION USED IN ALL ANALYTICAL AND EXPERIMENTAL ANALYSIS IS AS FOLLOWS: POSITIVE (+) FOR MOMENTS THAT PLACE THE INNER SURFACES OF THE FRAME IN TENSION AND NEGATIVE (-) IF THE MOMENTS PLACED THE INNER FACE OF THE FRAME IN COMPRESSION. FOR SHEARING FORCES A POSITIVE (+) SHEARING FORCE WAS CONSIDERED TO BE ONE THAT TENDED TO SHEAR THE LOWER PORTION OF A VERTICAL MEMBER INWARD AND NEGATIVE (-) IF IT TENDED TO SHEAR IT OUTWARD. FOR THE SLOPING MEMBERS OF THE FRAME, A POSITIVE (+) SHEARING FORCE TENDED TO SHEAR THE OUTER PORTION UPWARD AND NEGATIVE (-) SHEAR TENDED TO SHEAR IT DOWNWARD.

THE MAXIMUM EXPECTED SNOW LOAD AND WIND LOADS USED WERE OBTAINED FROM <u>UNITED STATES NAVY TECHNICAL PUBLICATION, NAVDOCS TP-TE-3.</u>

A maximum snow load of 45 pounds per square foot of horizontal projected area was used which is adequate for nearly all parts of the United States. For wind loads, a maximum wind speed of 90 mph was used which is adequate for the entire United States under normal conditions excepting a few coastal regions. The velocity pressure was found from the following equation which was obtained from the previously mentioned publication.

$$P = 0.00256\ V^2$$

P = force in pounds per square foot

V = velocity in miles per hour

The pressure coefficients and interior air pressures were also obtained from this publication and are listed as follows:

roof windward = -.20

roof leeward = -.50

wall leeward = -.50

wall windward = +.75

The roof coefficients are for the 1 to 3 slope used in this analysis. A negative (-) sign indicates an outward force, and positive (+) an inward force. For interior air pressures the following coefficients were considered: If there is an opening on the windward side, use +0.6P normal to all interior surfaces and -0.4P if there is an opening on the leeward side of the building.

Grain loads were also considered since structures of this type would likely be used for storage of grain. For these conditions it was assumed that a retaining wall would be placed inside the vertical member of the frame and supported by the frame as a distributed, varying load. See Figure 6. It was also assumed that the grain level was at

THE HAUNCH OF THE FRAME. THE LATERAL PRESSURE AGAINST THE FRAME AT ANY POINT BELOW THE GRAIN LEVEL WAS FOUND BY USE OF THE FOLLOWING EQUATION, (2):

$$L = WY \tan^2 (45^\circ - \frac{\phi}{2})$$

IN WHICH,

L = UNIT LATERAL PRESSURE, LB/FT^2

W = WEIGHT OF MATERIAL, LB/FT^3

Y = DEPTH OF MATERIAL, FT

ϕ = ANGLE WHOSE TANGENT EQUALS THE COEFFICIENT OF FRICTION BETWEEN GRANULES OF THE MATERIAL

FOR THESE CALCULATIONS WHEAT WAS USED, WHICH HAS A UNIT WEIGHT OF 49 LB/FT^3 AND AN ANGLE OF REPOSE OF 25 DEGREES.

FOR CALCULATION PURPOSES, THE DEAD LOAD DUE TO THE STRUCTURAL MATERIAL WAS NEGLECTED SINCE IT PROBABLY WOULD BE SMALL COMPARED TO THE OTHER LOADS CONSIDERED. THE THREE LOADING CONDITIONS AND THEIR APPLICATIONS TO THE FRAME ALONG WITH FRAME DIMENSIONS ARE SHOWN IN FIGURE 6.

FOR EVALUATION PURPOSES A UNIT LENGTH, 1 FT, OF THE BUILDING WAS USED. TO OBTAIN THE MAXIMUM EXPECTED STRESSES, A COMBINATION OF THE VARIOUS LOADS WERE CONSIDERED AS SUGGESTED IN THE UNITED STATES NAVY BULLETIN MENTIONED PREVIOUSLY. THE VALUE OF THESE STRESSES PER UNIT OF BUILDING LENGTH FOR THE TWO TYPES OF FRAMES CONSIDERED ARE TABULATED IN TABLE II. THE BENDING MOMENT AND SHEAR DIAGRAMS FOR THE THREE MAJOR TYPES OF LOADING ARE PRESENTED IN FIGURES 7, 8, AND 9.

THE TABULATED VALUES IN TABLE II REVEAL THAT IN NEARLY ALL CASES, THE BENDING MOMENTS AT THE HAUNCH AND PEAK OF THE FRAME WERE LESS FOR THE HINGELESS FRAME AS COMPARED TO THE TWO-HINGED FRAME.

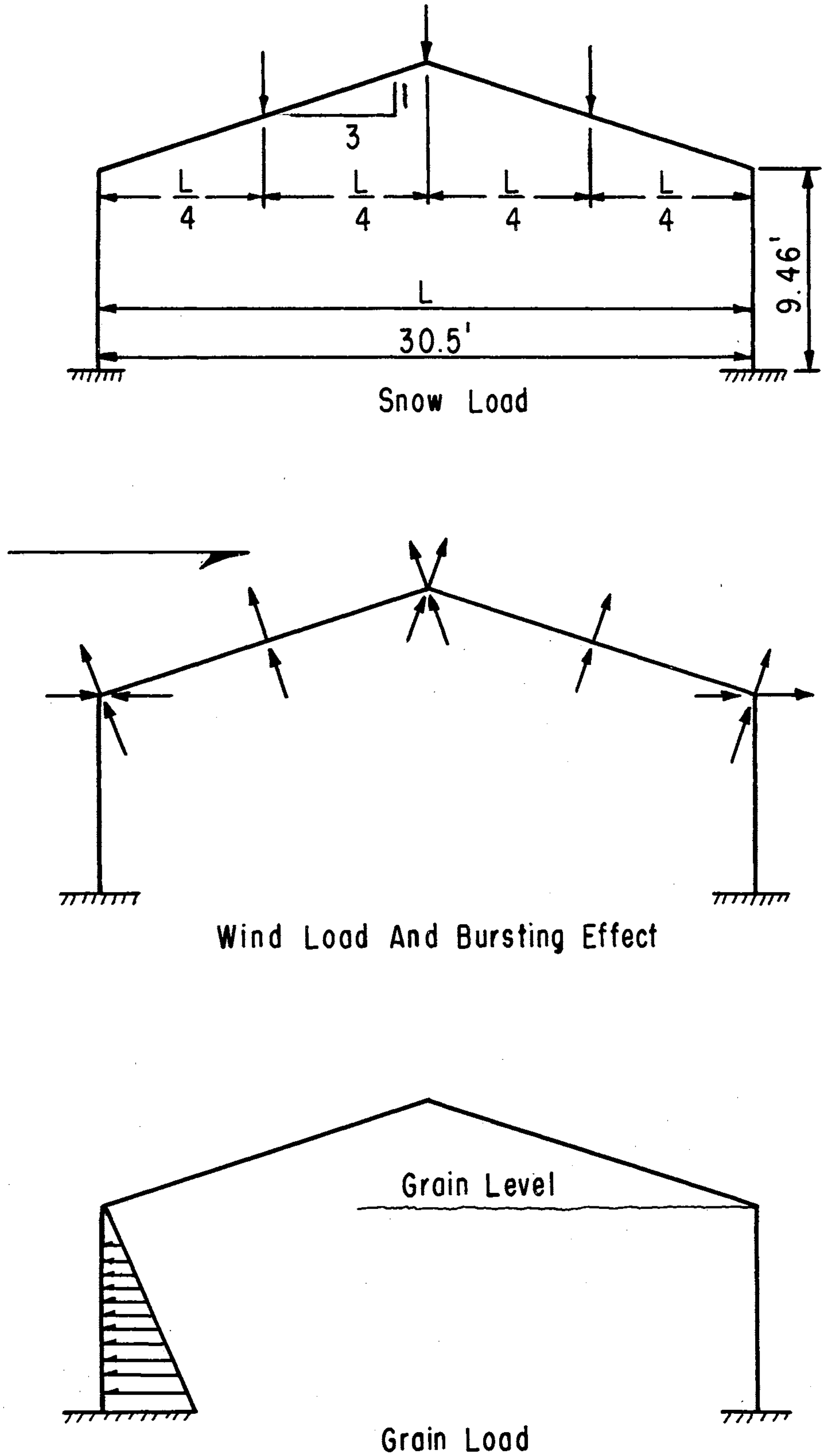


FIGURE 6. APPLICATION OF LOADS

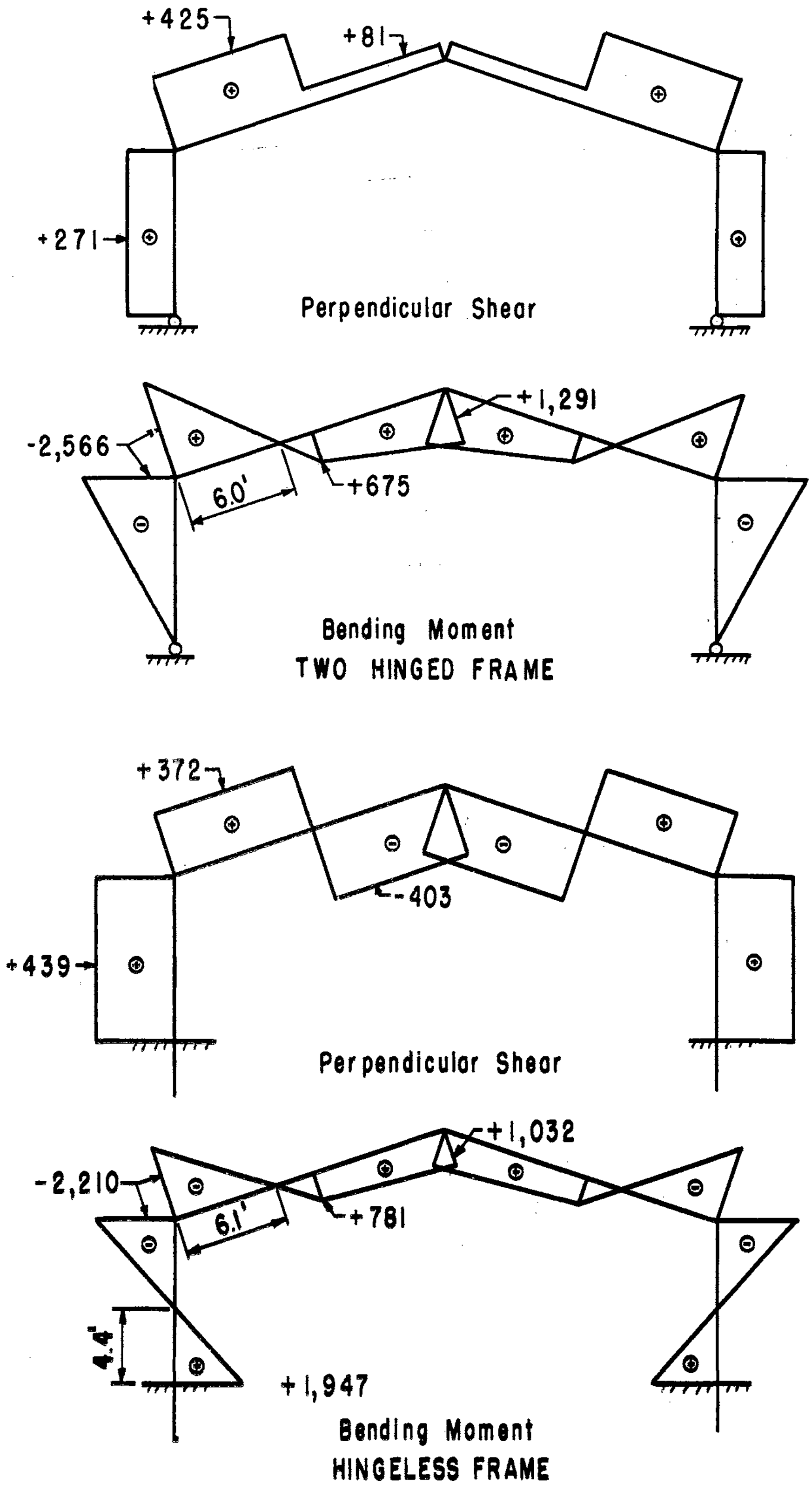


FIGURE 7. SNOW LOAD, SHEAR AND BENDING MOMENT DIAGRAMS

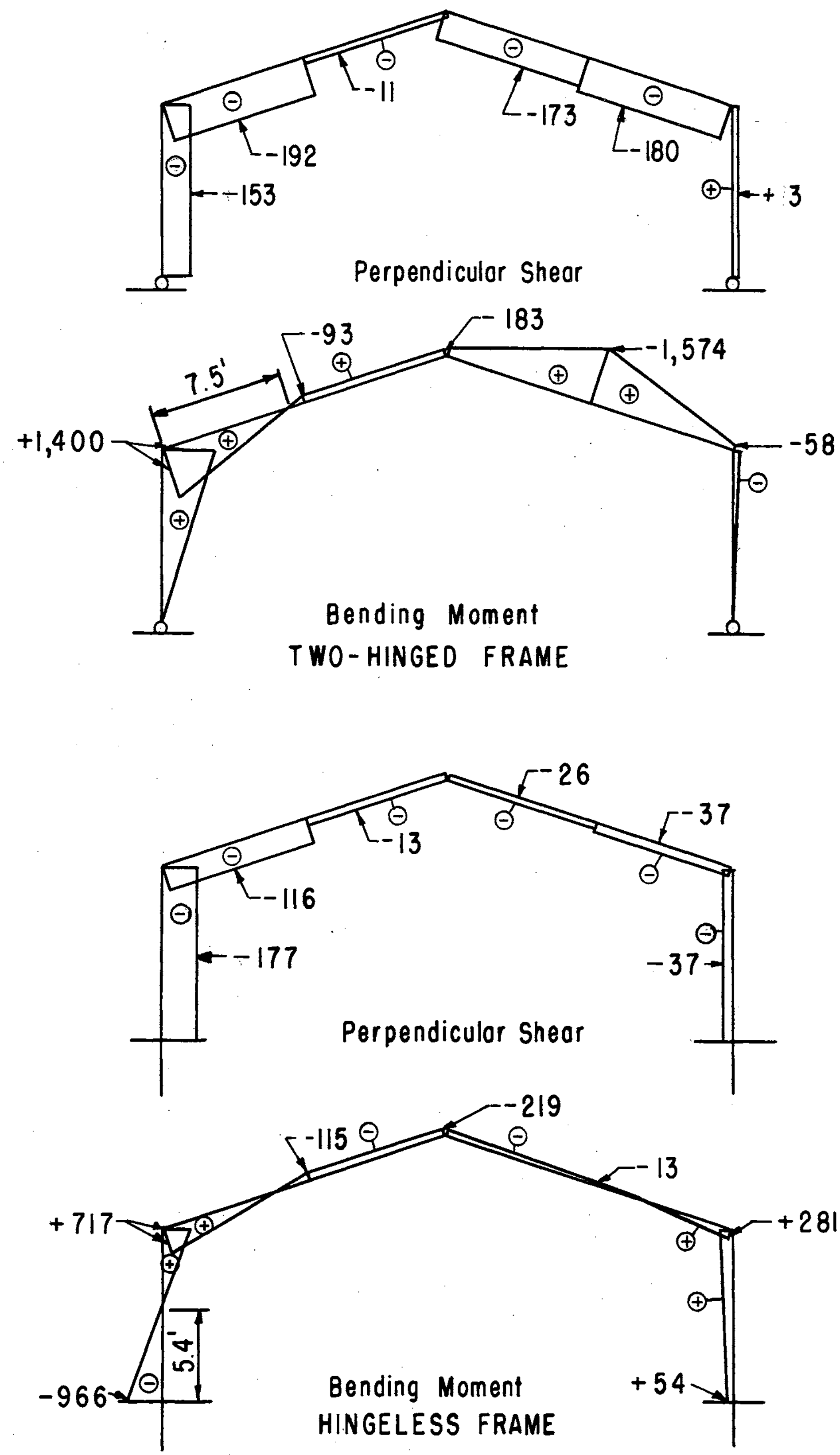


FIGURE 8. WIND LOAD AND INTERNAL PRESSURE, SHEAR AND BENDING MOMENT DIAGRAMS

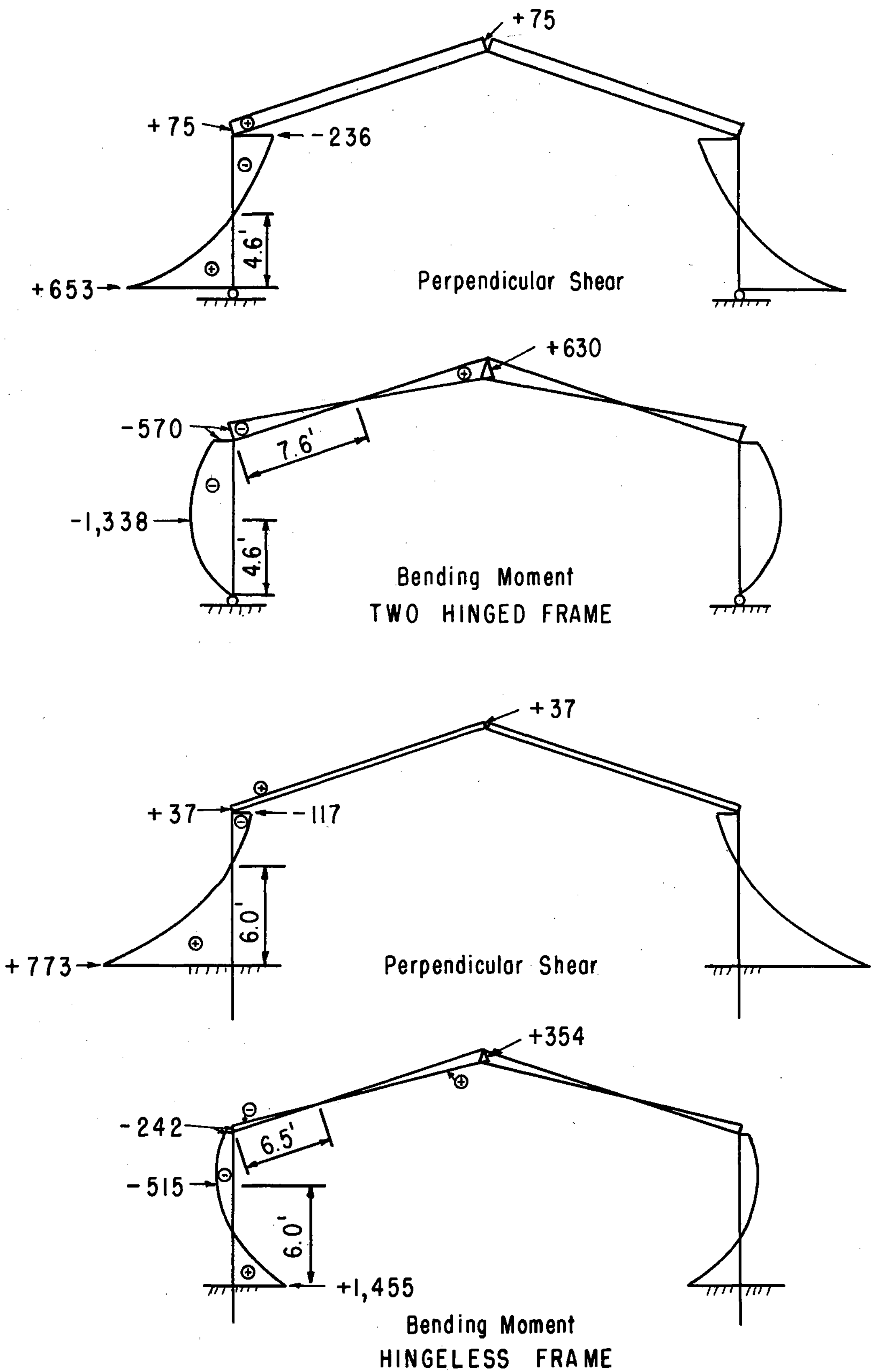


FIGURE 9. GRAIN LOAD, SHEAR AND BENDING MOMENT DIAGRAMS

TABLE II

STRESSES DUE TO VARIOUS EXPECTED LOADING CONDITIONS
IN HINGELESS AND TWO-HINGED FRAMES

Position		M_A	M_B	M_C	M_D	M_E	H_A	H_E	V_A	V_E
Snow Load	HL	+1,976	-2,097	+948	-2,097	+1,976	+431	+431	+515	+515
	TH	0	-2,565	+1,291	-2,565	0	+271	+271	+515	+515
Wind Load + Internal Pressure (outward)	HL	-966	+717	-219	+281	+54	-177	-23	-264	-187
	TH	0	+1,400	-183	-58	0	-153	+3	-264	-187
Grain Load	HL	+1,455	-242	+354	-242	+1,455	+773	+773	0	0
	TH	0	-570	+630	-570	0	+653	+653	0	0
Snow Load + Grain Load	HL	+3,431	-2,339	+1,302	-2,339	+3,431	+1,204	+1,204	+515	+515
	TH	0	-3,135	+1,921	-3,135	0	+924	+924	+515	+515
1/2 Wind Load + Snow Load + Grain Load	HL	+2,948	-1,981	+1,192	-2,198	+3,458	+1,116	+1,192	+431	+493
	TH	0	-2,435	+1,829	-3,164	0	+847	+926	+389	+422
Wind Load + 1/2 Snow Load + Grain Load	HL	+1,533	-563	+584	-999	+2,553	+817	+971	+90	+212
	TH	0	-506	+959	-2,278	0	+641	+797	+4	+71
1/2 Wind Load + Snow Load	HL	+1,493	-1,739	+838	-1,956	+2,003	+342	420	+431	+493
	TH	0	-1,865	+1,199	-2,594	0	+194	+273	+389	+422
Wind Load + 1/2 Snow Load	HL	+77	-321	+341	-757	-1,097	+38	+193	+90	+212
	TH	0	-662	+261	-1,394	0	-12	+144	+4	+71

Note: HL = Hingeless Frame TH = Two-Hinged Frame M = Bend Moment H = Horizontal Force
V = Vertical Force

IT WAS ALSO SHOWN THAT OF THE THREE MAIN TYPES OF LOADING CONSIDERED, SNOW LOAD PRODUCED THE MAXIMUM BENDING MOMENTS.

THE REASON THE HAUNCH AND PEAK OF THE FRAME ARE OF MOST INTEREST IS THAT AT THESE POINTS, THE FRAME MEMBERS ARE USUALLY FASTENED TOGETHER BY SOME TYPE OF JOINT. ALSO, MAXIMUM MOMENTS USUALLY OCCUR AT THE HAUNCH. THESE JOINTS COULD THEN BE CONSIDERED POINTS OF LIKELY FAILURE. IN THE CONSTRUCTION OF A LIGHT STRUCTURAL HINGELESS-FRAME, THE SIDE MEMBERS ARE USUALLY EXTENDED BELOW THE GROUND LEVEL TO OBTAIN THE HINGELESS-FRAME CONDITION. FOR THIS REASON, THE POINTS AT THE GROUND LEVEL WOULD PROBABLY BE LESS LIKELY TO FAIL THAN THE HAUNCH OR CROWN JOINTS. ALSO IN A HINGELESS FRAME, THERE WOULD LIKELY BE SOME DEFORMATION OF SOIL WHICH WOULD TEND TO RELIEVE STRESSES AT THESE POINTS.

OF THE VARIOUS LOADING CONDITIONS CONSIDERED, IT APPEARS THAT THE SNOW LOAD PLUS GRAIN LOAD WOULD BE THE CONDITION TO DESIGN FOR SINCE IT PRODUCES MAXIMUM BENDING MOMENTS. FOR EXPERIMENTAL PURPOSES, HOWEVER, THE SNOW LOADING CONDITIONS WOULD PROBABLY BE ADEQUATE FOR TESTING PURPOSES SINCE IT DEVELOPED UP TO 90 PER CENT FOR HINGELESS, AND 82 PER CENT FOR TWO-HINGED OF THE MAXIMUM STRESSES AT THE HAUNCH THAT WERE DEVELOPED BY THE LOADING CONDITION WHICH PRODUCED THE MAXIMUM STRESSES. IT IS ALSO QUITE EASY TO SIMULATE THE GRAVITY LOADS IN THE LABORATORY, WHEREAS GRAIN LOADS WOULD BE VERY DIFFICULT TO SIMULATE.

THE SNOW LOAD USED IN THESE COMPUTATIONS IS QUITE HIGH. HOWEVER, IF THIS TYPE OF STRUCTURE WERE DESIGNED FOR CONSTRUCTION ANY WHERE IN THE UNITED STATES, IT WOULD SEEM QUITE REASONABLE TO USE THESE LOADS. IF THE STRUCTURE WERE DESIGNED FOR A CERTAIN AREA, THESE LOADS SHOULD then be adjusted for that area.

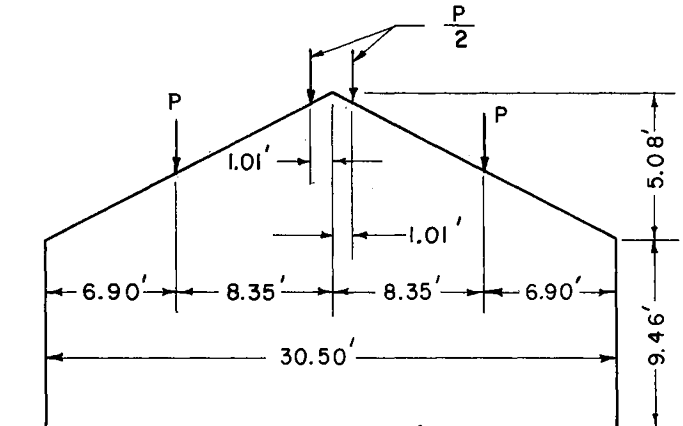
It was observed in these computations that the shearing forces normal to the structural members were considerably higher in the hingeless frame. However, the magnitude did not appear to be high enough to be of great concern.

Theoretical Deflection and Stresses

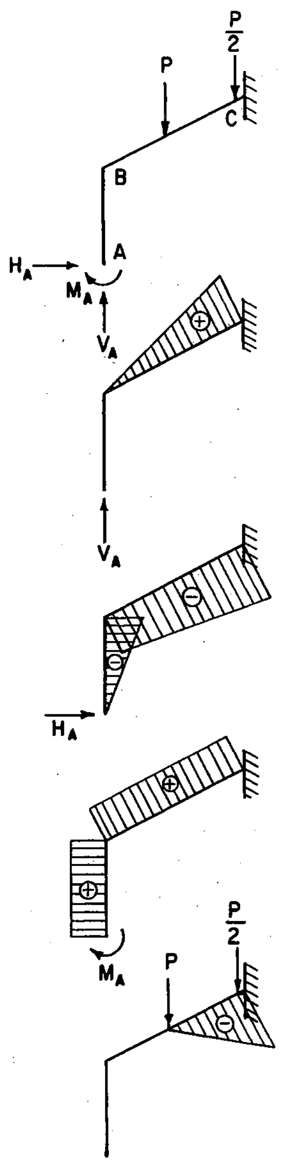
The purpose of the analytical investigation was to predict as accurately as possible the deflections and stresses in the frames to be tested. Therefore, calculations were made using the actual dimensions and points of loading of the frames to be tested. The method used for this analysis was the moment-area method since it can be used to find both stresses and deflections.

In order to accurately predict these deflections and stresses, however, one must also have a reasonably accurate value for the modulus of elasticity and the moment of inertia of the frame section. This is often referred to as the EI value or the stiffness of a structural member. These values can be obtained from various sources. However, it was deemed necessary in this case to determine an EI value by conducting experiments on actual frame members to obtain a more reliable value. An experiment was then set up to obtain this value for the frames to be tested. This experiment is explained in more detail in Chapter IV.

The next step was to calculate the stresses and deflections in each frame type and loading conditions applied to it. As an example, the hingeless frame with gravity loading conditions is considered. The calculations are shown in Figure 10. For these calculations the



Actual Loading Conditions Of Frame



Hingeless Frame, Gravity Load

1. Moment Area Equations Used

$$\Delta \phi = \int_C^A \frac{M\,ds}{EI} = 0 \qquad \Delta X = \int_C^A Y_A \frac{M\,ds}{EI} = 0$$

Assume P = 1 lb.

2. Equating $\Delta \phi = 0$

$$\Delta \phi = \frac{1}{EI} \int_C^A M\,ds = 0$$

Parts Of Integral

Due to V_A = 183.81 ft.^{2} – lb.
Due to H_A = – 237.59 ft.^{2} X H_A
Due to M_A = 25.53 ft. X M_A
Due to Loads = – 36.98 ft.^{2} – lb.

$$\Delta \phi = \frac{1}{EI}\left[25.53\,M_A - 237.586\,H_A + 146.06\right] = 0$$

3. Equating $\Delta X = 0$

$$\Delta X = \frac{1}{EI} \int_C^A Y_A\,M\,ds = 0$$

Parts Of Integral

Due to V_A = 2,361.29 ft.^{2} – lb.
Due to H_A = – 2,630.85 ft^2 X H_A
Due to M_A = 238.06 ft. X M_A
Due to Load = – 503.70 ft.^{2} – lb.

$$\Delta X = \frac{1}{EI}\left[238.06\,M_A - 2{,}630.85\,H_A + 1857.59\right] = 0$$

4. Solving for H_A and M_A

$\Delta \phi = M_A - 9.31\,H_A + 5.76 = 0$
$\Delta X = M_A - 11.05\,H_A + 7.80 = 0$
H_A = 1.171 lb./lb. of Load
M_A = 5.133 ft.- lb./lb. of Load

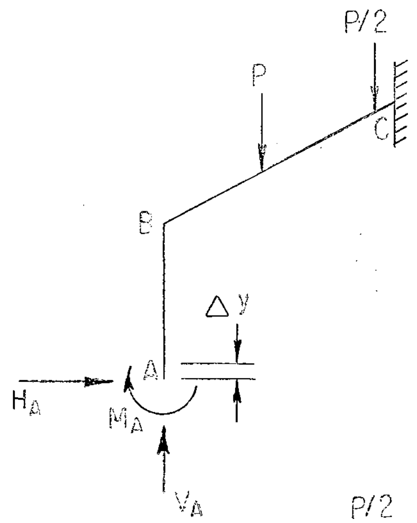
FIGURE 10. THEORETICAL STRESSES IN HINGELESS FRAME

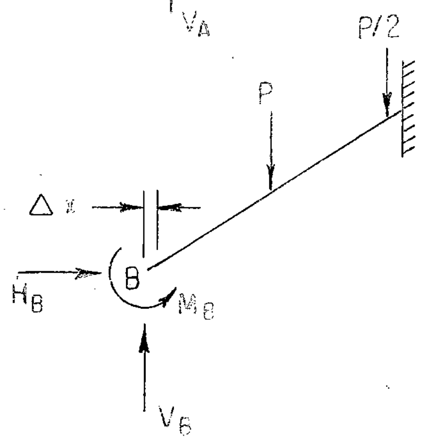
crown was considered to be fixed since the frame and its loading conditions were symmetric about that point. The only values not known at point A on the frame were H_A and M_A. Therefore, two moment-area equations were needed to solve for these values. With these values known, the bending moments and axial loads can be evaluated at any point in the frames by the use of equations from statics. This was done for each position on the frame where the stresses were to be experimentally determined. The next step was to determine the deflections at the crown and haunch. These calculations are shown in Figure 11. In this case as in the previous one, the crown was assumed to be fixed. These calculations were all carried out assuming P to be equal to one pound of load. The weight of the frame itself was neglected.

Analytical calculations for the two-hinged frame were carried out in a similar manner. The only difference was that only one moment-area equation was needed since the horizontal force was the only unknown at point A in the frame.

Next, analytical calculations were made to determine the effects of support movement. For the hingeless frame, pure rotation and translational support movement in a lateral direction were considered. Lateral support movement only was considered for the two-hinged frames. Calculations for determining the stresses in the hingeless frame due to support movements are shown in Figure 12. Again the crown was assumed to be fixed against rotation due to the symmetry of the frame and loading. To get the symmetric loading, both supports were considered to move the same amount but in opposite directions.

HINGELESS FRAME
DEFLECTIONS DUE TO GRAVITY LOADS





1. ASSUME P = 1 POUND
   THEN FROM PREVIOUS CALCULATIONS
   $H_A = 1.171$ LB/LB OF LOAD
   $M_A = 5.133$ FT-LB/LB OF LOAD

2. DEFLECTION AT CROWN

$$\Delta Y = \int_C^A X_A \frac{M\,ds}{EI} = \frac{1}{EI} \int_C^A X_A \, M\,ds$$

PARTS OF INTEGRAL

DUE TO V_A	=	1,868.6 FT^3-LB
DUE TO H_A	=	1,842.6 FT^3-LB
DUE TO M_A	=	628.9 FT^3-LB
DUE TO LOAD	=	-466.0 FT^3-LB

$$\Delta Y = \frac{1}{EI}\left[189.0 \text{ FT-LB}\right] \text{/LB OF LOAD}$$

3. DEFLECTION AT HAUNCH

$$\Delta X = \int_C^B Y_B \frac{M\,ds}{EI} = \frac{1}{EI} \int_C^B Y_B \, M\,ds$$

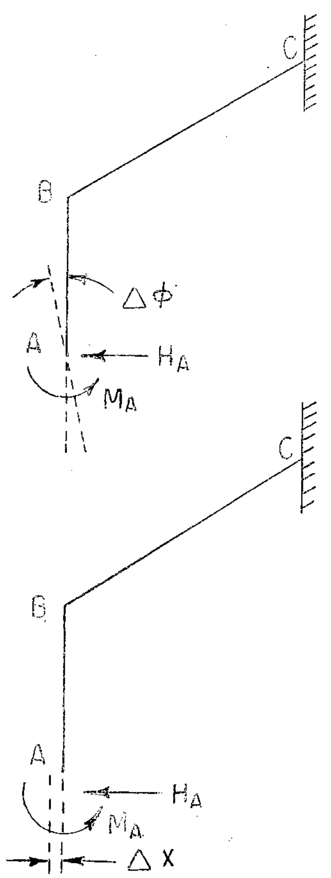
PARTS OF INTEGRAL

DUE TO V_B	=	622.5 FT^3-LB
DUE TO H_B	=	-161.8 FT^3-LB
DUE TO M_B	=	-242.5 FT^3-LB
DUE TO LOAD	=	-155.7 FT^3-LB

$$\Delta X = \frac{1}{EI}\left[62.5 \text{ FT}^3\text{-LB}\right] \text{/LB. OF LOAD}$$

FIGURE 11. DEFLECTIONS IN HINGELESS FRAMES.

HINGELESS FRAME
Stress Due to Support Movement



1. Equations Used

$$\Delta\phi = \int_C^A \frac{M\,ds}{EI}, \quad \Delta x = \int_C^A \frac{M\,ds}{EI}$$

From previous calculations

$$\Delta\phi = \frac{1}{EI}\left[25.53\,M_A - 237.59\,H_A\right]$$

$$\Delta x = \frac{1}{EI}\left[238.06\,M_A - 2{,}630.85\,H_A\right]$$

2. Support Rotation

Assume $\Delta\phi = -.01$ radians

$$\Delta\phi = \frac{1}{EI}\left[25.53\,M_A - 237.59\,H_A\right] = .01$$

$$\Delta x = \frac{1}{EI}\left[238.06\,M_A - 2{,}630.85\,H_A\right] = 0$$

$$H_A = EI \times (2.244 \times 10^{-4})$$

$$M_A = EI \times (2.480 \times 10^{-3})$$

3. Translational Movement

Assume $\Delta x = .01$ feet outward

$$\Delta\phi = \frac{1}{EI}\left[25.53\,M_A - 237.59\,H_A\right] = 0$$

$$\Delta x = \frac{1}{EI}\left[238.06\,M_A - 2{,}630.85\,H_A\right] = .01$$

$$H_A = EI \times (2.407 \times 10^{-5})$$

$$M_A = EI \times (2.240 \times 10^{-4})$$

Figure 12. Stresses in hingeless frames due to support movement

Deflections were found in the same manner as for the gravity-loading conditions. Calculations to determine the stresses and deflections for the two-hinged frame due to lateral support movement are similar to those for translational support movement of the hingeless frame. The only difference is that there is no bending moment at point A and E of the frame.

Values of the theoretically determined stresses and deflections for each frame and loading conditions are listed in Appendix B along with the experimentally determined stresses and deflections.

CHAPTER IV

EXPERIMENTAL STUDY

Experimental Design

Preliminary Tests

For the theoretical analysis, an experimentally determined value of EI was used for determining the theoretical stresses and deflections in the frames to be tested.

For this determination of EI, a straight section of the test frame was used. In determining the length of section to be tested, the following two problems had to be considered: (1) Make the section as long as possible to obtain high bending moments and low perpendicular-shearing stresses, and (2) Use a section short enough to have lateral stability so that lateral bracing would not have to be used, which might affect the results.

This experiment was also set up to determine accuracy and reliability of electrical resistant gages for determining stresses and bending moments in the test frame. For this purpose strain gages were mounted on either side of the test section as shown in Figure 13. This part of the experiment also served as a check for the EI value that was to be determined by measuring the deflections in the test section.

The next problem concerned the number of sections to test and the

NUMBER OF CYCLES TO RUN ON EACH TEST SECTION. FOR THIS EXPERIMENT, IT APPEARED THAT AS MANY TEST SECTIONS AS POSSIBLE SHOULD BE USED TO DETERMINE THE AMOUNT OF DIFFERENCE IN THE TEST SECTIONS DUE TO NON-UNIFORMITY OF THE SECTIONS.

FOR THIS EXPERIMENT, FOUR TEST SECTIONS WERE USED WITH TWO LOADING CYCLES PER TEST SECTION. EACH LOADING CYCLE CONSISTED OF A LOADING AND UNLOADING PHASE. USING THIS EXPERIMENTAL SETUP, THE VARIANCE DUE TO TEST SECTIONS, LOADING CYCLES, AND THE LOADING AND UNLOADING PHASE OF EACH CYCLE COULD BE COMPUTED, WHICH PROVIDED HELPFUL INFORMATION IN DESIGNING THE FRAME EXPERIMENTS.

FRAME TESTS

FOR THE FRAME EXPERIMENTS, GRAVITY LOADS WERE SIMULATED. THE DECISION TO USE GRAVITY LOAD WOULD PROBABLY APPEAR OBVIOUS WHEN THE AMOUNT OF EQUIPMENT AND TIME INVOLVED IN SIMULATING WIND AND GRAIN LOADS ARE CONSIDERED. FOR THE GRAVITY LOADING CONDITIONS THE LOADS WERE APPLIED AT POSITIONS ON THE FRAME WHERE THE PURLINS WOULD ACTUALLY BE PLACED IN CONSTRUCTION.

NEXT, THE PROBLEM OF SELECTING THE POSITIONS ON THE FRAME AT WHICH DEFLECTION AND STRAIN MEASUREMENTS WERE TO BE MADE WAS CONSIDERED. FOR DEFLECTION MEASUREMENTS, THE PEAK AND HAUNCH DEFLECTIONS APPEARED TO BE OF MOST IMPORTANCE AND WERE THEREFORE USED. THE DECISION WHERE TO MAKE STRAIN MEASURMENTS WAS A MORE ARBITRARY ONE SINCE THE MEASUREMENTS COULD NOT BE MADE DIRECTLY AT THE PEAK OR HAUNCH DUE TO THE FRAME CONSTRUCTION. IT IS QUITE EVIDENT, HOWEVER, THAT THEY NEEDED TO BE AS CLOSE TO THE PEAK AND

HAUNCH AS POSSIBLE BECAUSE OF THE HIGH BENDING MOMENTS AT THESE POINTS. THE STRAIN GAGES WERE PLACED 1 1/2 FEET ON EITHER SIDE OF THE HAUNCH AND PEAK. THIS WAS TO MOVE THE GAGES FAR ENOUGH AWAY FROM THE JOINT SO THAT THE JOINT STRUCTURE WOULD NOT EFFECT THE STRAIN READINGS. GAGES WERE ALSO PLACED ON THE LOWER PORTION OF THE SIDE MEMBERS WHICH WAS A POINT OF HIGH BENDING MOMENT IN THE HINGELESS FRAME. FOR THE HINGELESS FRAMES, THESE GAGES WERE PLACED 6 INCHES ABOVE THE GROUND LEVEL AND FOR THE TWO-HINGED FRAMES THEY WERE PLACED 1 FOOT ABOVE THE GROUND LEVEL. THE REASON FOR MOVING THE GAGES UP ON THE TWO-HINGED FRAMES WAS THAT THE BENDING MOMENT IS THEORETICALLY ZERO AT THE GROUND LEVEL. THEREFORE, THEY WERE MOVED UP TO A POSITION WHERE ENOUGH STRAIN WOULD BE PRODUCED TO ALLOW MEASUREMENT WITH SOME DEGREE OF ACCURACY.

THE NEXT DECISION WAS WHERE THE GAGES SHOULD BE PLACED ON THE FRAME MEMBERS IN REFERENCE TO THE CROSS-SECTION CONFIGURATION. FOR THE PRELIMINARY TESTS, A GAGE WAS PLACED ON EACH C-SHAPED SECTION AND ON OPPOSITE SIDES OF THE MEMBER. THE RESULTS FROM THESE PRELIMINARY TESTS SHOWED A LARGE VARIANCE RATIO DUE TO THE POSITION OF THE GAGES. THEREFORE, FOR THE FRAME TEST THE GAGES WERE PLACED ONE ON EITHER SIDE OF ONE C-SHAPED SECTION AS SHOWN IN FIGURE 13.

FROM AN EVALUATION OF THE PRELIMINARY TESTS, IT WAS FOUND THAT THERE WAS A LARGE AMOUNT OF VARIANCE DUE TO THE TEST SECTIONS WHEN MEASURING DEFLECTION. THEREFORE, AS MANY FRAMES AS POSSIBLE WERE TESTED AND FEWER TEST CYCLES RUN. FOR THESE TESTS THERE WERE FOUR HINGELESS AND FOUR TWO-HINGED FRAMES AVAILABLE AND THEY WERE ALL TESTED IN ORDER TO DETERMINE VARIATION AMONG THEM AS ACCURATELY AS POSSIBLE.

THE FOLLOWING TESTS WERE THEN MADE:

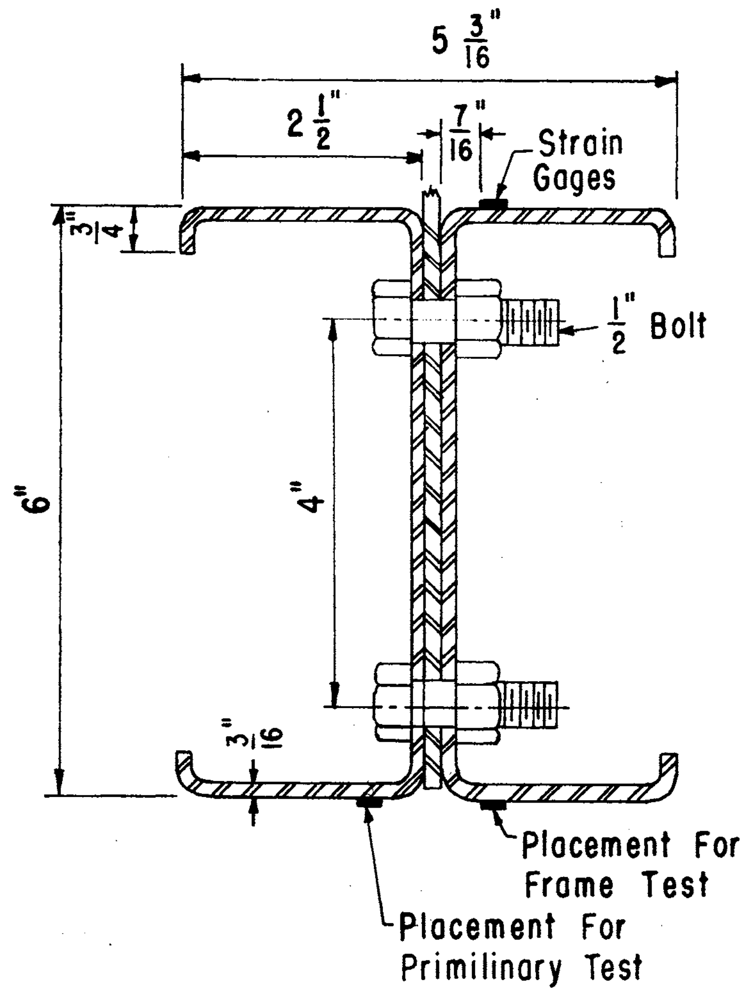


FIGURE 13. CROSS-SECTION OF FRAME MEMBER AND POSITION OF STRAIN GAGES.

(1) gravity loads, fixed supports

(2) support rotation only

(3) translational support movement only

For two-hinged frames

(1) gravity loads, pinned-end supports

(2) gravity loads, restrained supports

(3) lateral support movement only

Experimental Equipment

These experiments were conducted in the Agricultural Engineering Light Structures Laboratory which has a floor expecially made for conducting tests such as these. This floor is a 5 1/2 inch thick concrete floor with steel channels spaced every two feet apart to which brackets or braces can be bolted directly.

The frames used for these experiments were designed and manufactured by Armco Drainage and Metal Products, Inc. These frames were made of 0.100 inch thick cold-formed steel which had a yield strength of approximately 37,000 psi. Gusset plates used at the haunch and peak were 3/16 inch thick steel plates with a two inch leg bent up for added stiffness. A complete sketch of the frame with its dimensions is shown in Figure 14. Bolts used to assemble the frames were 1/2 inch, high-strength steel bolts.

The preliminary experiments to obtain an EI value of the frame section were conducted using straight sections of the frame. These sections were supported on either end by two heavily constructed H-shaped supports spaced 12 feet apart. To apply the loads, a hydraulic cylinder was used which could be activated by either a motor driven pump or by high pressure inert gas acting through

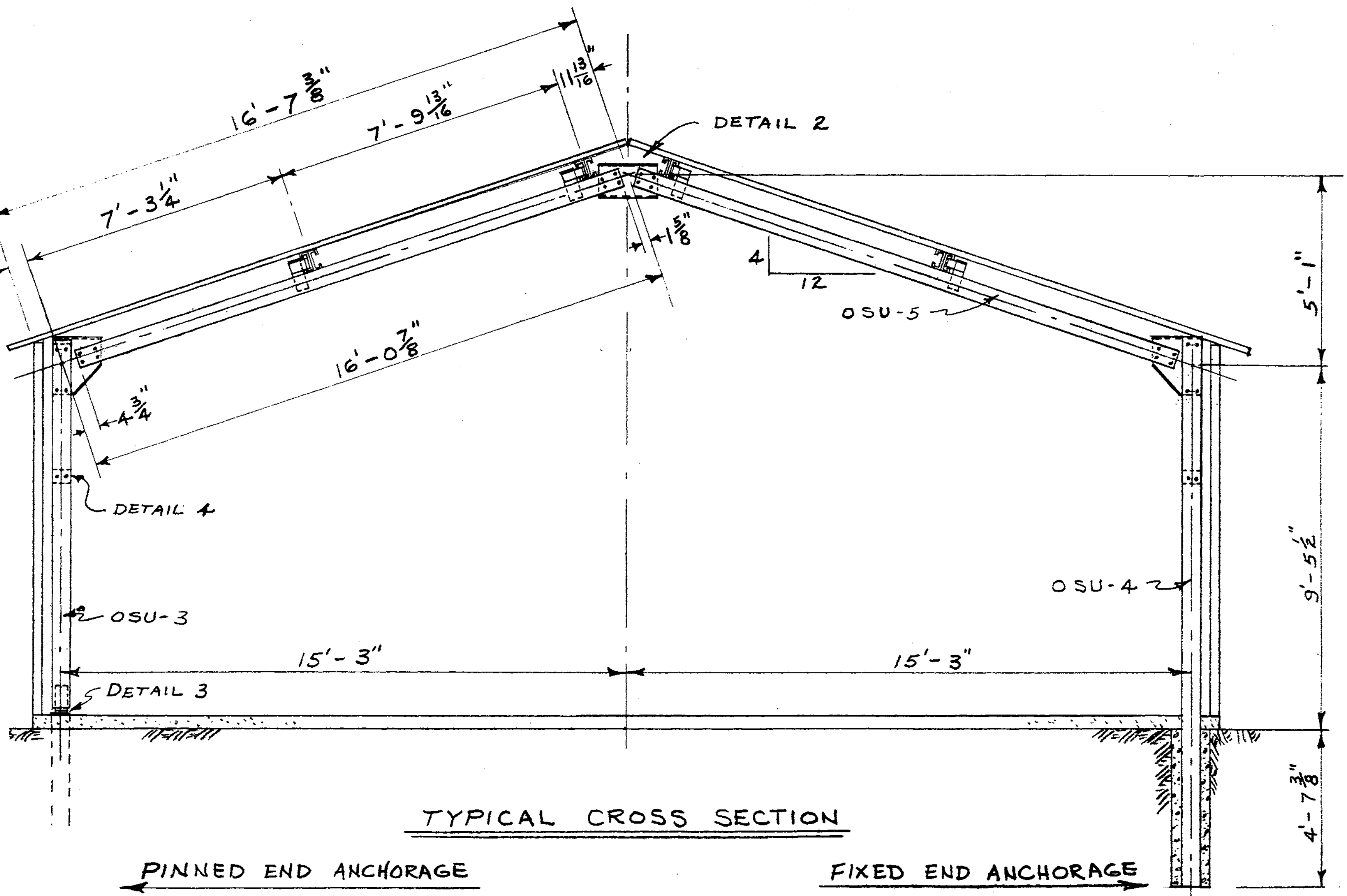


Figure 14. Over-all sketch of frame.

AN ACCUMULATOR. TO APPLY THE LOADS DIRECTLY TO THE FRAME SECTION, AN INVERTED A-SHAPED BRACKET WAS USED. THIS BRACKET WAS USED TO DIVIDE LOAD EVENLY BETWEEN TWO POINTS OF APPLICATION AND PLACED THE PORTION BETWEEN THESE POINTS AT A CONSTANT BENDING MOMENT. TO MEASURE THE APPLIED LOAD, A LOAD LINK WAS PLACED BETWEEN THE A-SHAPED BRACKET AND HYDRAULIC CYLINDER. THEN TO MEASURE THE STRAIN IN THE LOAD LINK, WHICH WAS CALIBRATED AGAINST LOAD, A BALDWIN STRAIN INDICATOR WAS USED. THIS INDICATOR WAS ALSO USED TO MEASURE THE STRAIN PICKED UP BY THE STRAIN GAGES MOUNTED ON THE TEST SECTIONS. THE STRAIN GAGES USED WERE 1/2 INCH, 120 OHM RESISTANCE METAL FOIL GAGES. FOR THE DEFLECTION MEASUREMENTS, A DIAL MICROMETER WAS USED.

BRACKETS TO HOLD THE SUPPORT ENDS OF THE HINGELESS FRAMES WERE MADE OF 5 INCH X 1 3/4 INCH CHANNELS. TO ADD STIFFNESS TO THE BRACKETS, A 5/16 INCH X 1 1/2 INCH METAL PLATE WAS WELDED IN THE MIDDLE OF EACH CHANNEL. THIS GAVE THE SUPPORT END OF THE FRAME AN EI VALUE APPROXIMATELY 7.4 TIMES AS LARGE AS FOR THE EI VALUE OF THE FRAME ITSELF. THESE BRACKETS WERE CONSTRUCTED AS SHOWN IN FIGURE 15, AND AN INSTALLATION OF ONE IS SHOWN IN FIGURE 16. THE BRACKETS WERE SUPPORTED IN SUCH A MANNER THAT THEY COULD BE ADJUSTED LATERALLY IN ANY MANNER DESIRED BY SIMPLY ADJUSTING THE BOLTS THAT HELD THEM IN PLACE.

SUPPORT BRACKETS FOR THE TWO-HINGED FRAMES WERE CONSTRUCTED TO GIVE A PINNED-END CONDITION. THESE SAME BRACKETS WERE ALSO CONSTRUCTED SO THE SUPPORT END COULD BE FIXED IF DESIRED. PROVISIONS FOR LATERAL SUPPORT MOVEMENT WERE ALSO MADE. CONSTRUCTION DETAILS ARE GIVEN IN FIGURE 18 AND TYPICAL INSTALLATIONS ARE SHOWN IN FIGURES 17 A, AND 17 B. FIGURE 17 A SHOWS A TYPICAL PINNED CONDITION AND FIGURE 17 B SHOWS AN INSTALLATION IN WHICH THE END WAS FIXED.

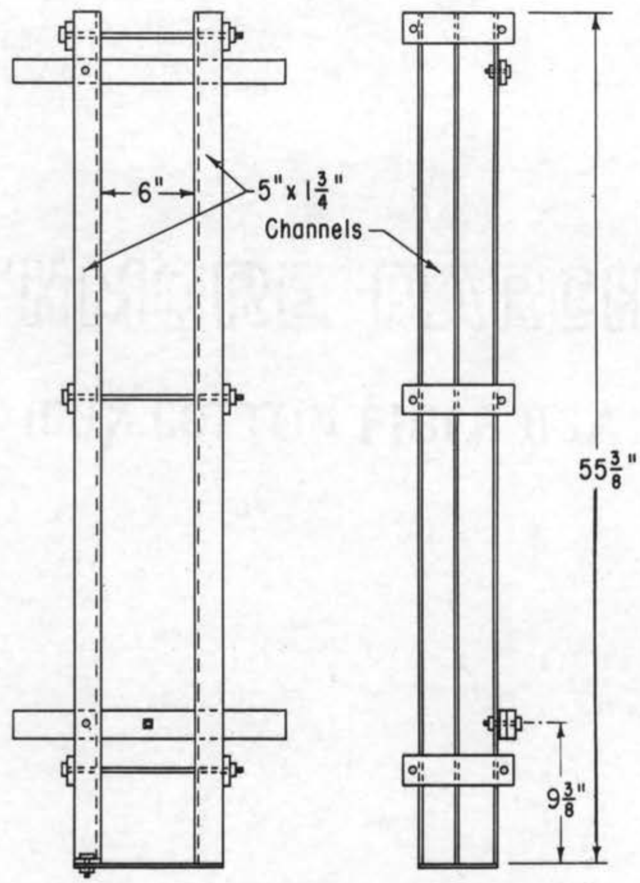


FIGURE 15. BRACKETS FOR HINGELESS END CONDITION

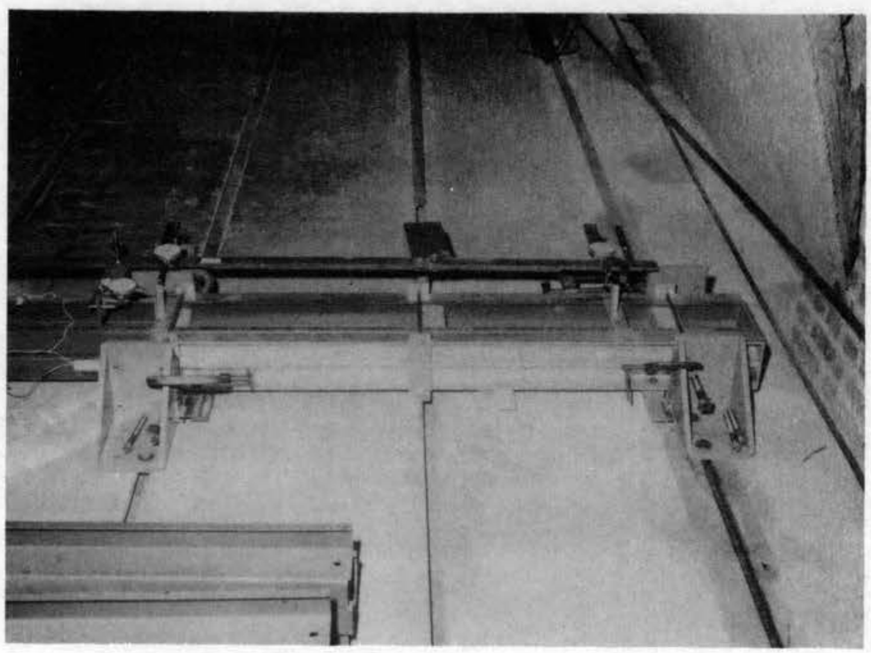


FIGURE 16. INSTALLATION OF HINGELESS END BRACKETS.

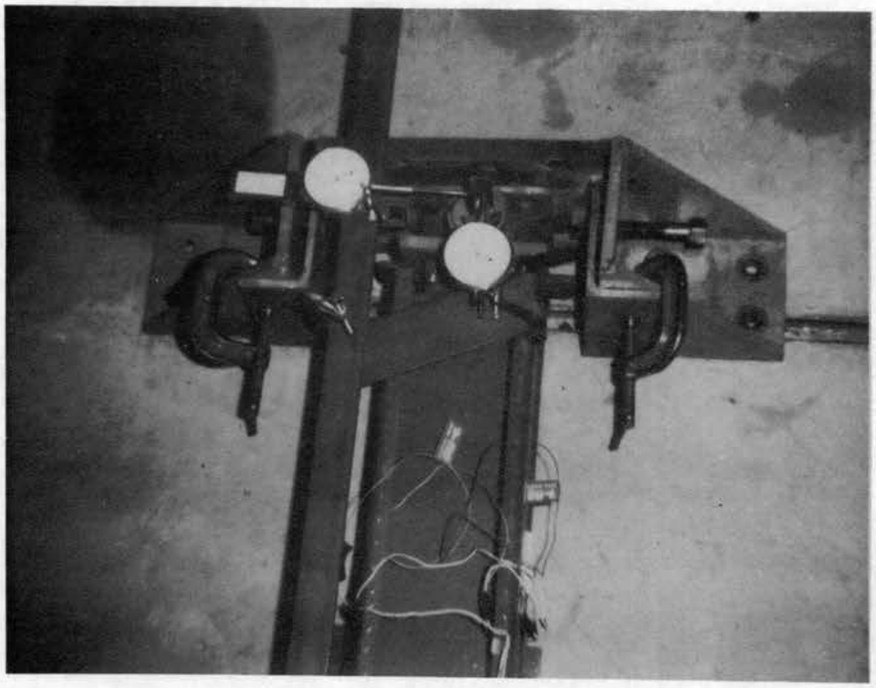


Figure 17A. Installation of two-hinged brackets as a pinned end condition.

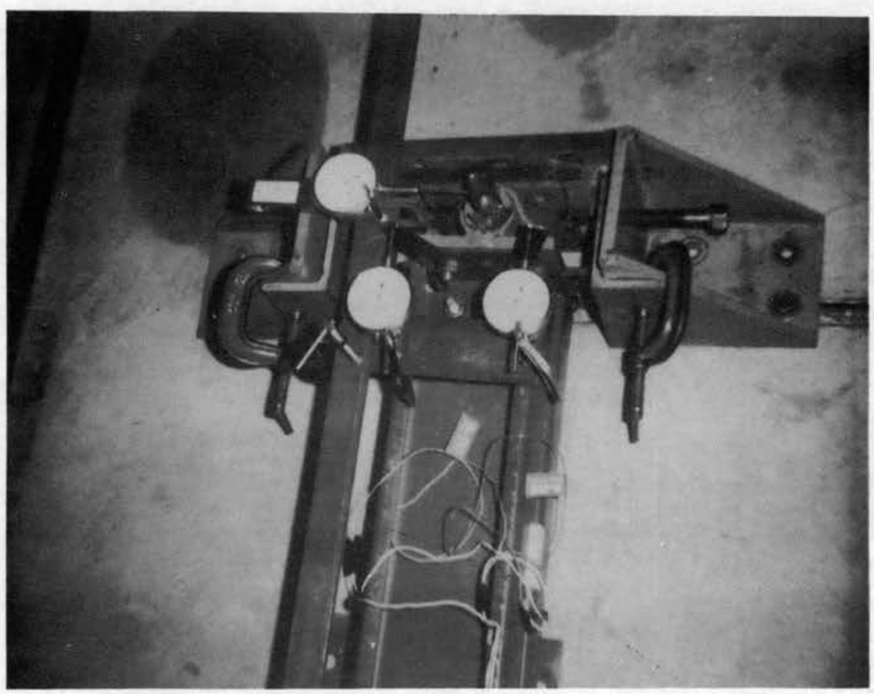


Figure 17B. Installation of two-hinged brackets with end of support fixed.

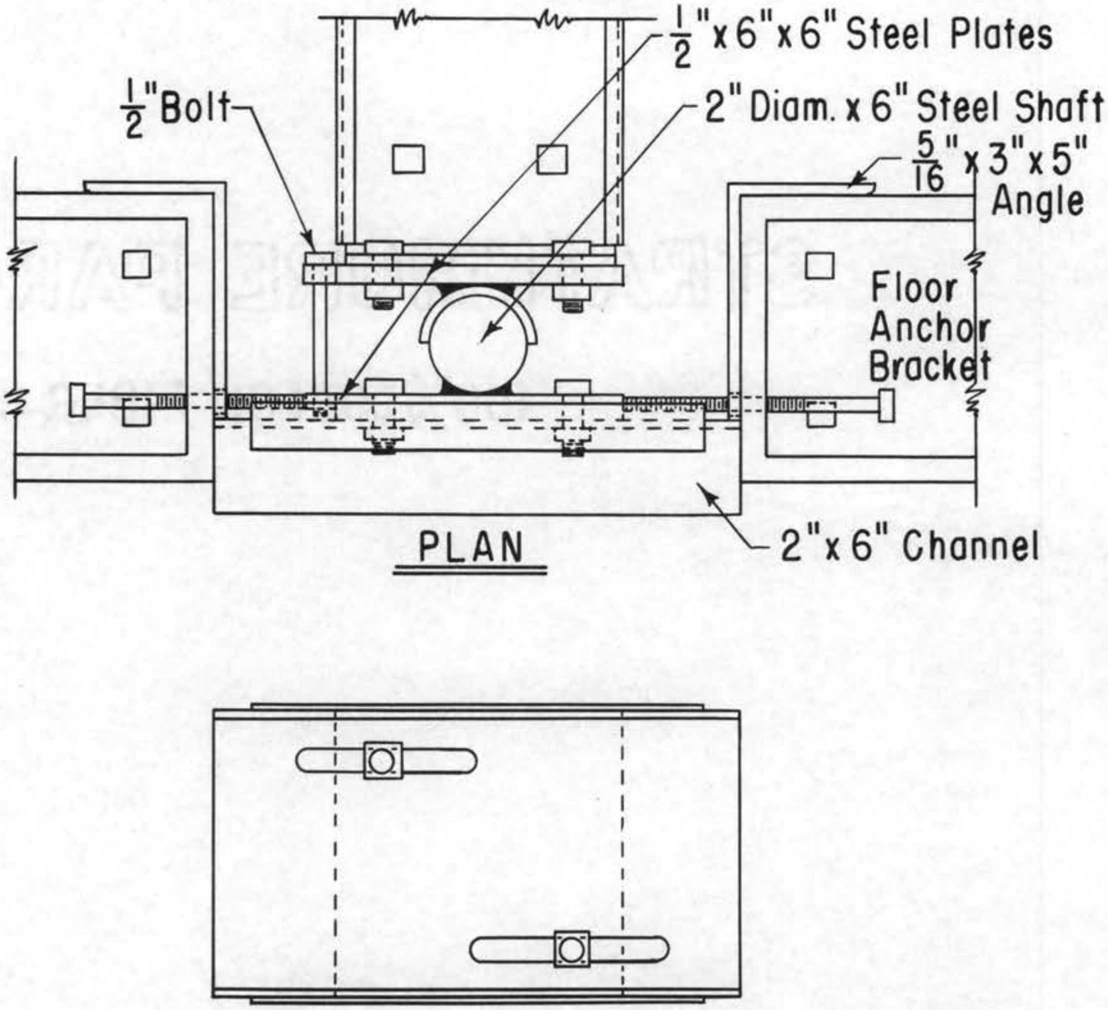


FIGURE 18. BRACKETS FOR TWO-HINGED CONDITION

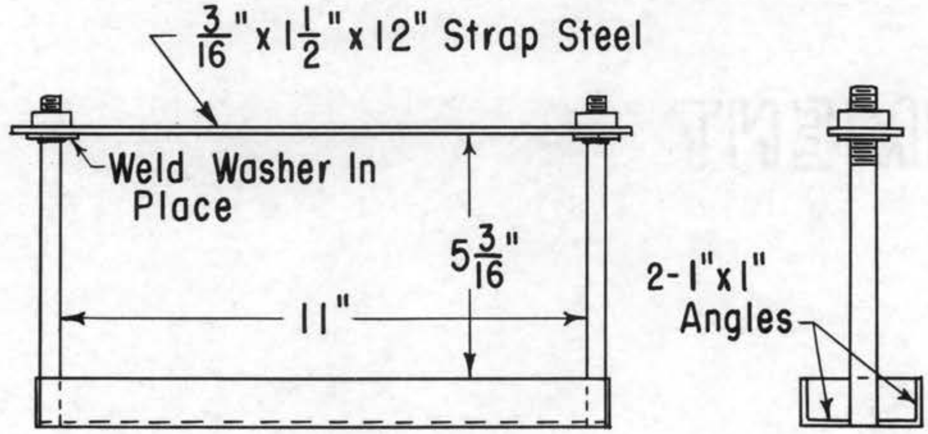


FIGURE 19. BRACKETS USED FOR LATERAL SUPPORT

The brackets used to give the frame lateral support while testing are shown in Figure 19. This support would be provided by the frame covering and other lateral bracing in actual construction. The positions at which these brackets were placed is shown in Figure 20.

To apply the gravity loads three matched hydraulic cylinders were used. These cylinders were activated by the same pressure source used for the preliminary experiments. The cylinder loads were then applied to the frames as shown in Figure 20.

Measurement of the loads applied to the frame was accomplished by use of a pressure cell. This pressure cell was made from two 3 1/2 inch high pressure pipe flanges. A metal plate was placed between the two flanges on which four strain gages were mounted. Hydraulic pressure was applied to one side of the plate and strain measurements due to bulging were made on the other side. A diagram of the pressure cell is shown in Figure 21. To measure the actual force applied by the cylinders, a Baldwin Type U-1 load cell was used.

Strain gages used to measure strain in the frames were Baldwin-Lima-Hamilton Corporation (FA-100-12) etched foil gages. These gages had a length of one inch and 120 ohms resistance. To measure deflection, dial micrometers were used.

To determine the possibility of the frame members rotating with respect to the gusset plate at the haunch, a measuring system was constructed at that point. For this purpose two 1/8 inch welding rods were bent to the proper shape and glued directly to the frame members with the free ends extending beyond the members. They were placed in a position so that they would rotate about the center of the bolt spacings. Rotation was measured with a ruler placed on a board

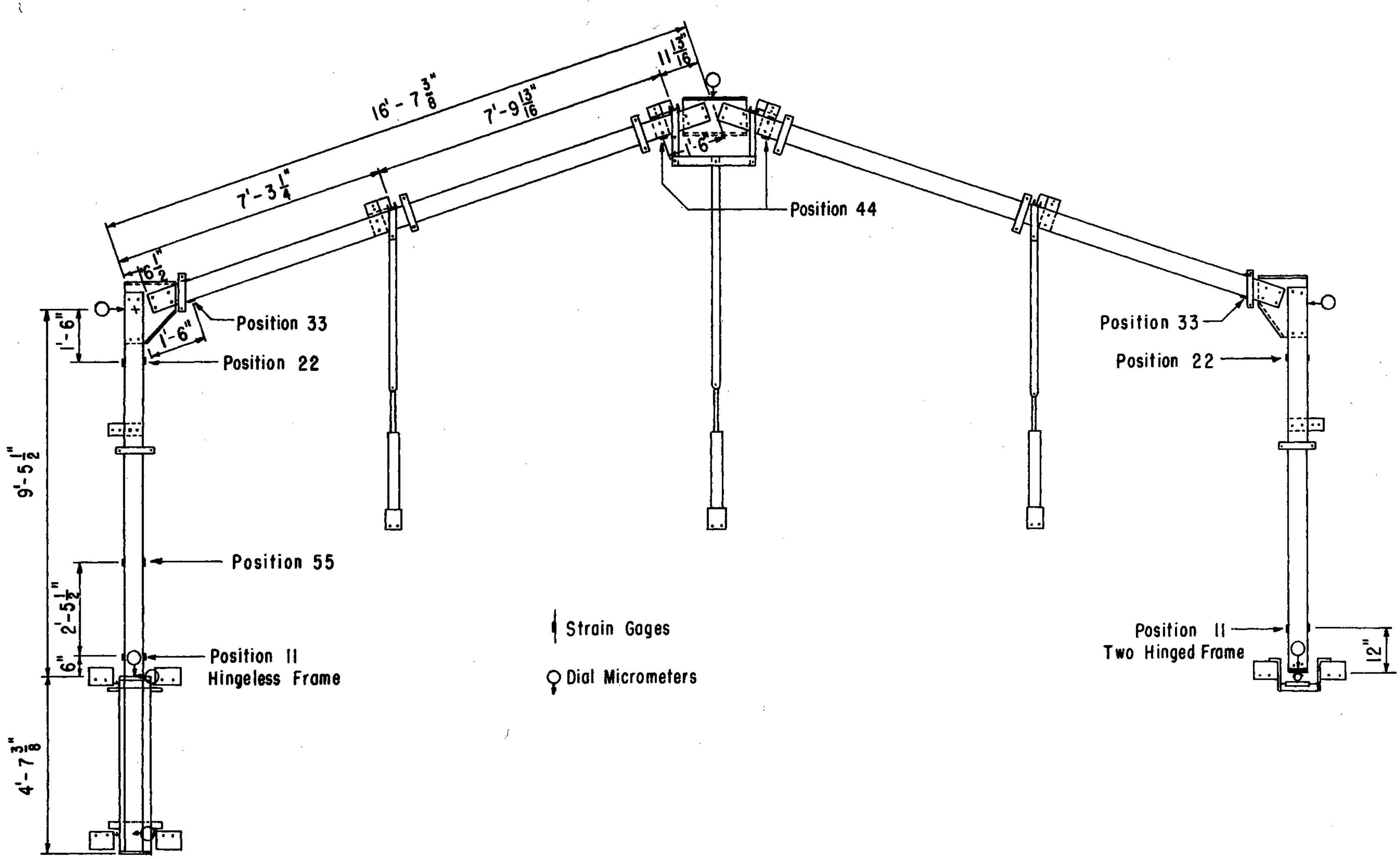


FIGURE 20. DIAGRAM SHOWING EXPERIMENT EQUIPMENT IN PLACE AND POSITIONS OF DEFLECTIONS AND STRAIN MEASUREMENTS.

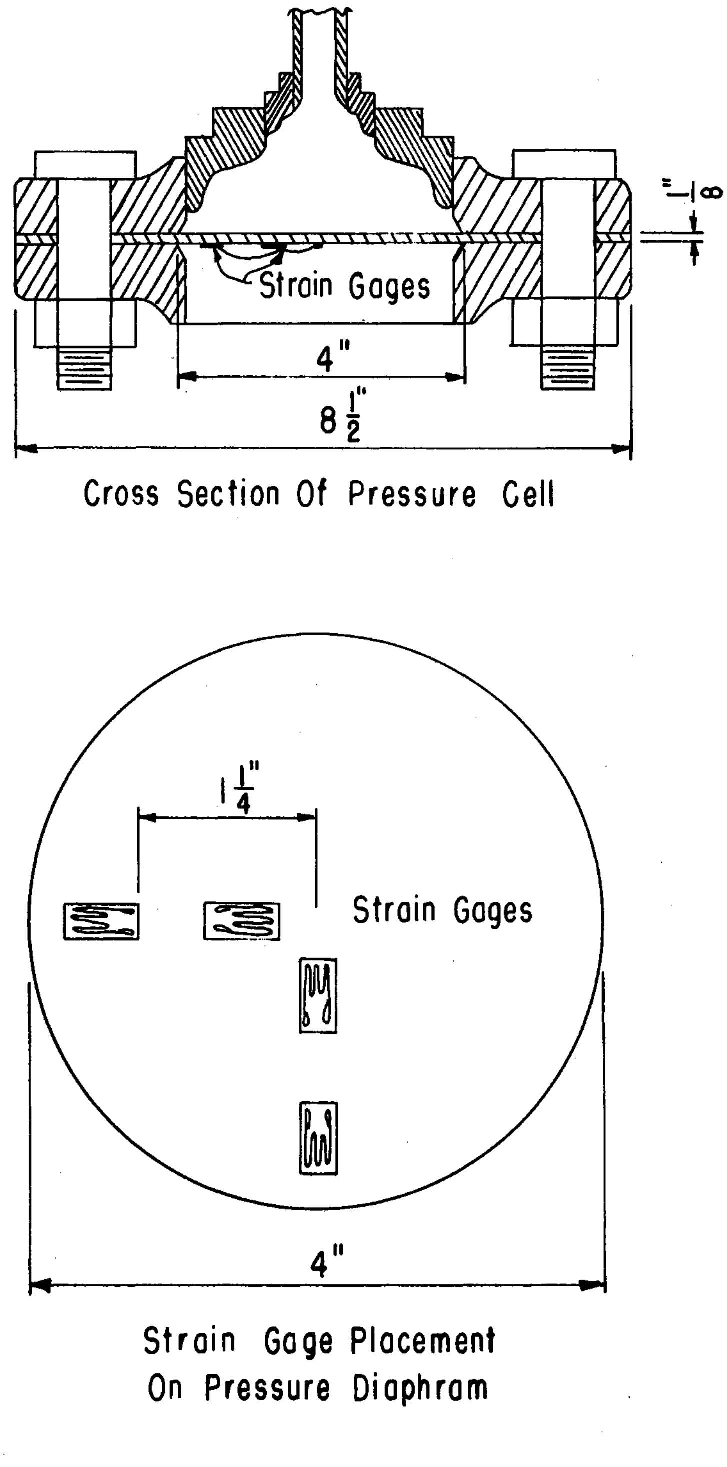


FIGURE 21. PRESSURE CELL USED TO MEASURE HYDRAULIC PRESSURE.

attached directly to the gusset plate. By this means rotation of either member could be measured with respect to the gusset plate. This setup is shown in Figure 22.

The entire installation of equipment ready for testing is shown in Figure 23.

Experimental Procedures

Preliminary Tests

When testing the frame members, the first step was to place the members between the supports and then clamp and bolt plates between the member sections to keep them from twisting under load. Next, these members were preloaded with a load as great or larger than the test load, and this load was then left on for two or three minutes. After preloading, zero strain, deflection, and load link readings were taken. Loads were then applied in approximately six equal increments. Strain, deflection, and load link readings were taken for each load increment. To start the unloading cycle, a load greater than the load for which readings were taken during the loading cycle, was applied and then unloaded to approximately this same load. Unloading was then carried out in the same manner as the loading cycle with six equal increments of unloading being used. All subsequent tests were carried out in the same manner.

Frame Tests

To determine applied loads, each cylinder was calibrated in terms of applied load and hydraulic pressure applied. For this

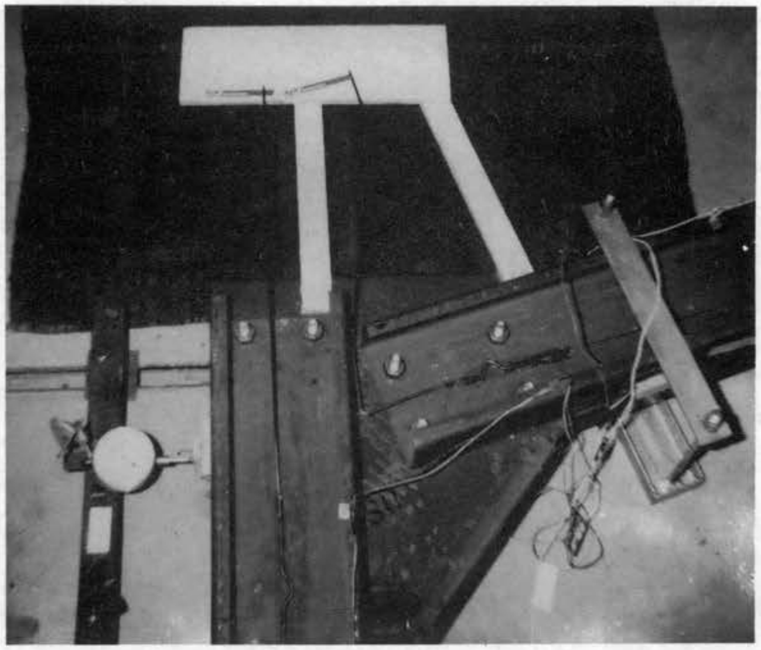


FIGURE 22. ROTATION MEASUREMENT SETUP.

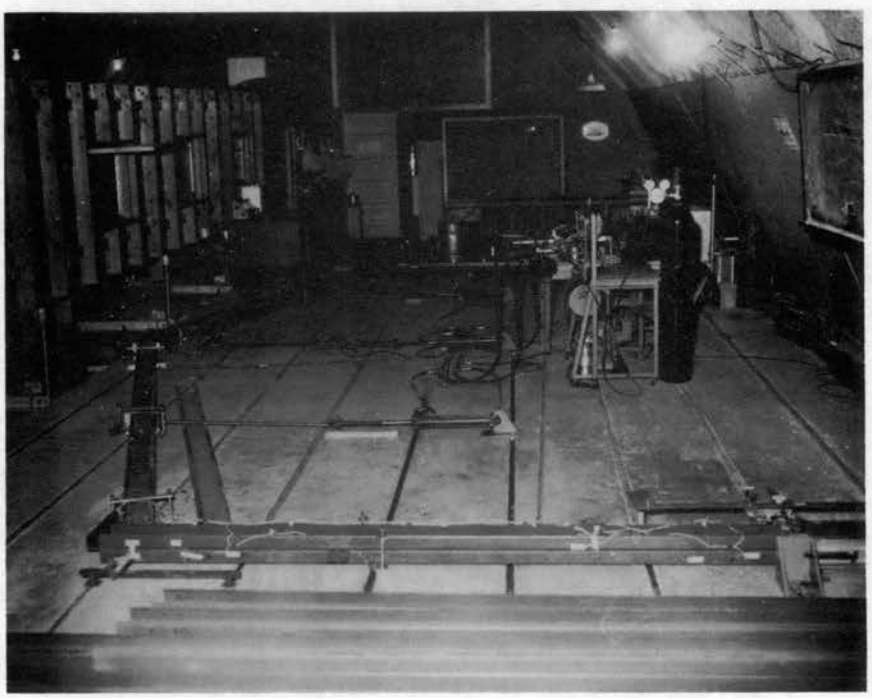


FIGURE 23. INSTALLATION OF EQUIPMENT READY FOR TESTING.

CALIBRATION, EACH CYLINDER WAS ATTACHED TO A LOAD CELL WHICH MEASURED THE LOAD APPLIED IN POUNDS. TO MEASURE THE HYDRAULIC PRESSURE, A PRESSURE CELL WAS USED AND THE STRAIN READING OBTAINED FROM THE CELL WAS CALIBRATED IN TERMS OF LOAD APPLIED BY THE CYLINDER.

BEFORE ASSEMBLING ANY FRAMES, ALL STRAIN GAGES WERE ATTACHED TO THE PROPER FRAME MEMBERS, WHICH WERE SELECTED AT RANDOM. THIS WAS DONE MOSTLY FOR CONVENIENCE AND ALSO TO ALLOW THE STRAIN GAGE CEMENT TO DRY THOROUGHLY BEFORE USE.

DURING FRAME ASSEMBLY, THE BOLTS FASTENING THE FRAME MEMBERS TOGETHER WERE TIGHTENED ONLY ENOUGH TO GET A SNUG FIT. WHEN ALL BRACKETS WERE IN PLACE AND THE FRAME IN TEST POSITION, THE BOLTS WERE TIGHTENED WITH A TORQUE OF 105 FT-LB OR GREATER. THEN AT LEAST THREE GRAVITY LOADING CYCLES WERE APPLIED AND THE BOLTS AGAIN TIGHTENED TO TAKE UP ANY SLACK DUE TO JOINT MOVEMENT. ALSO DURING THE PRELOADING CYCLES, ROTATION MEASUREMENTS OF THE FRAME MEMBERS WITH RESPECT TO THE GUSSET PLATES AT THE HAUNCH WERE NOTED. IF THE ROTATION APPEARED TO BE OF SIGNIFICANCE, IT WAS RECORDED. NEXT, ZERO READINGS WERE TAKEN FOR THE STRAIN GAGES, DIAL MICROMETERS, AND PRESSURE CELL. LOADS WERE THEN APPLIED IN EQUAL INCREMENTS UNTIL THE MAXIMUM THEORETICALLY DETERMINED LOAD WAS REACHED. FOR THE FIRST HINGELESS FRAME TESTED, AN ATTEMPT WAS MADE TO RUN A LOADING AND UNLOADING PORTION FOR EACH CYCLE. HOWEVER, THE UNLOADING RESULTS WERE SO POOR THAT ONLY THE LOADING PORTION FOR ALL SUBSEQUENT TESTS WERE RUN. AFTER EACH INCREMENT OF LOAD WAS APPLIED, THE SUPPORTS WERE ADJUSTED TO THE ORIGINAL POSITIONS BEFORE ANY READINGS WERE TAKEN. THIS WAS TO REDUCE ANY EFFECT THAT SUPPORT MOVEMENT MIGHT HAVE ON THE RESULTS.

After the gravity loads were applied, the support movement tests were conducted. Support rotation of the hingeless frames was carried out by rotating the supports about the point which represented the ground level. To do this, the top of the support was held in a fixed position and the bottom rotated inward for both supports at the same time. This rotation was measured by dial micrometers placed at a known distance from the top and recorded in terms of radians rotation per support. The procedure used for conducting these tests was the same as for gravity loads, except the loading was carried out in increments or rotation rather than increments of applied load.

Translational support movement tests were conducted by moving both supports outward simultaneously. This movement was measured with dial micrometers and recorded as outward movement in feet of movement per support. These tests were then conducted as the previous tests with increments of outward movement.

Lateral support movement of the two-hinged frames was achieved by simply moving both supports outward simultaneously. This movement was measured with dial micrometers and recorded as feet of lateral movement per support.

Tests for determining the effect of fixing the two-hinged frame supports were conducted as follows: The support was adjusted for zero rotation and the two dial micrometers used for measuring rotation adjusted to the same readings. Then after each increment of gravity load, the dial micrometers were again adjusted to have the same reading by adjusting a bolt, Figure 18, used for rotation adjustment. Thus if the entire support moved in the direction of applied load, both dial micrometers would be moved the same amount and rotation could

STILL BE MEASURED. THIS CAN BE UNDERSTOOD MORE CLEARLY BY REFERENCE TO FIGURES 17 A AND 17 B.

TO TEST THE EFFECT OF EXTRA FASTENERS HOLDING THE TWO FRAME SECTIONS TOGETHER, FOUR ADDITIONAL FASTENINGS WERE USED IN THE TOP MEMBERS. THESE FASTENINGS WERE MADE BY DRILLING EXTRA HOLES IN THE MEMBERS AND FASTENING THEM TOGETHER WITH 1/2 INCH BOLTS, FIGURE 13. THESE FASTENINGS WERE MADE 1/3 OF THE DISTANCE BETWEEN ORIGINAL FASTENINGS INWARD FROM EACH END OF THE TOP MEMBERS. THIS WAS DONE FOR ONE TWO-HINGED FRAME ONLY. TO TEST THIS EFFECT, TWO GRAVITY LOAD CYCLES WERE RUN ON THE FRAME WITH THE EXTRA FASTENINGS.

FOR THE TEST TO DETERMINE THE EFFECT OF TIME VERSUS DEFLECTION, ONE TWO-HINGED FRAME WAS USED. A KNOWN LOAD WAS APPLIED AND HELD CONSTANT FOR THE DURATION OF THE TEST. FOR THIS TEST THE ONLY OBSERVATION USED WAS DEFLECTION TAKEN AT THE PEAK.

CHAPTER V

ANALYSIS OF DATA

To determine the EI value of the frame sections, the deflection readings obtained from preliminary tests were used. The value of deflection used was obtained by a linear regression analysis of the data from each cycle of each section tested. A mean value was then obtained from these regression coefficients which was used for the value of deflection. This value, .0827 inches per 1000 pounds of applied load, together with a moment-area equation was then used to obtain the EI value.

Regression analysis was performed by the least squares method. Regression coefficients refer to the slope of the line representing the relationship between load and deflection.

To investigate the variation in deflection that could be expected, an interval was found within which the mean deflection of other samples could be expected to fall 95 times out of 100. This interval was .1761 to .1893 inches deflection per 1000 pounds of applied load.

To serve as a check, the EI value was determined by strain data. The mean value of strain, obtained as in the preceeding analysis, was $.25125 \times 10^{-6}$ in/in per pound of applied load. E was estimated to be 29.5×10^{6} lb/in^2. Stress was then found by multiplying strain by E, which was 7.412 lb/in^2. Knowing the value of bending moment, 28.25

IN-LB/LB OF APPLIED LOAD, THE MOMENT OF INERTIA WAS THEN FOUND TO BE 11.24 IN^4 BY THE FOLLOWING EQUATION:

$$I = \frac{Mc}{F_B}$$

I = MOMENT OF INERTIA, IN^4

M = BENDING MOMENT, IN-LB

C = DISTANCE TO POINT OF STRAIN MEASUREMENT FROM NEUTRAL AXIS, IN

F_B = STRESS AT POINT OF MEASUREMENT, LB/IN^2

THE EI VALUE WAS FOUND BY MULTIPLYING THE COMPUTED I VALUE BY THE ASSUMED VALUE OF E.

STATISTICAL ANALYSIS OF VARIANCE, AS PRESENTED BY STEEL AND TORRIE (22), WAS APPLIED TO THE EXPERIMENTAL DATA. THE ANALYSIS OF VARIANCE TABLES ARE PRESENTED IN APPENDIX C.

COMPUTER PROGRAMS WERE WRITTEN TO DO A MAJOR PORTION OF THE DATA ANALYSIS FOR THE FRAME EXPERIMENTS. THESE PROGRAMS WERE WRITTEN IN FORTRAN, A COMPUTER PROGRAM WHICH VERY NEARLY REPRESENTS MATHEMATICAL EXPRESSIONS AND ARE LISTED IN APPENDIX A.

DEFLECTION AT THE PEAK WAS OBTAINED BY SUBTRACTING THE AVERAGE DEFLECTION OF BOTH SUPPORTS, MEASURED PARALLEL TO THE SUPPORTS FROM THE PEAK DEFLECTION. HAUNCH DEFLECTION WAS OBTAINED BY TAKING THE AVERAGE DEFLECTION OF BOTH HAUNCHES, MEASURED PERPENDICULAR TO THE COLUMN MEMBERS.

FROM THE STRAIN READINGS BOTH BENDING MOMENT AND AXIAL LOAD WERE CALCULATED. A GRAPHICAL REPRESENTATION OF THESE CALCULATIONS IS SHOWN IN FIGURE 24. THE STRAIN DUE TO AXIAL LOAD AND BENDING MOMENT WAS THEN MULTIPLIED BY A CONSTANT TO CONVERT THEM TO FT-LB BENDING MOMENT AND LB OF AXIAL LOAD.

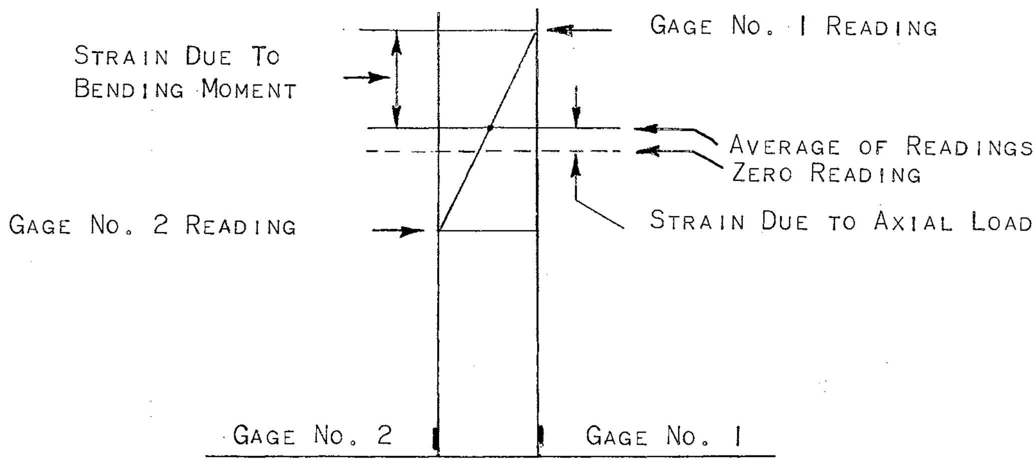


Figure 24. Graphical solution of strain readings.

Axial load was computed as follows:

$$P = E \epsilon A$$

in which P = load, lb

ϵ = measured strain, in/in

E = modulus of elasticity, lb/in^2

A = area of cross section, in^2

For these calculations, E and A were assumed to be 29.5×10^6 lb/in^2 and 2.34 in^2 respectively. To compute bending moment, the average value of strain per pound of applied load that was obtained from the preliminary test was used. This value was then divided into the bending moment produced for each pound of applied load. This value was

$$BM = \frac{2.354 \text{ ft-lb/lb of applied load}}{.25125 \text{ Micro inches of strain/lb of applied load}}$$

BM = 9.1708 FT-LB/MICRO INCH OF STRAIN

THE BENDING MOMENT AND AXIAL LOAD WERE COMPUTED FOR EACH POSITION AT WHICH TWO STRAIN GAGES WERE MOUNTED. TO REDUCE THE AMOUNT OF DATA, THE BENDING MOMENTS AND AXIAL LOADS AT SYMMETRICAL POSITIONS ON THE FRAME WERE AVERAGED. THIS ALSO HELPED TO AVERAGE OUT EFFECTS DUE TO UNEVEN LOADING OF THE FRAMES.

A REGRESSION ANALYSIS WAS APPLIED TO EACH SET OF READINGS TAKEN FOR EACH LOADING CYCLE. FROM THIS ANALYSIS, A VALUE OF A INTERCEPT ON Y AXIS, AND B, THE REGRESSION COEFFICIENT OR SLOPE OF THE LINE BEST FITTING THE DATA, WERE OBTAINED. ALSO DURING THIS OPERATION, A CORRELATION COEFFICIENT WAS COMPUTED, WHICH MEASURES THE DEGREE TO WHICH VARIABLES VARY TOGETHER OR A MEASURE OF THE INTENSITY OF ASSOCIATION (22). THESE PREVIOUSLY MENTIONED VALUES, ALONG WITH THE THEORETICAL DETERMINED VALUE OF B FOR EACH SET OF DATA, ARE LISTED IN APPENDIX B.

TO DETERMINE THE PROBABILITY OF OBTAINING A MEAN B VALUE LARGER OR SMALLER THAN THE THEORETICAL B VALUE, A VALUE OF T, STUDENT'S T, (22) WAS COMPUTED. THIS VALUE

$$T = \frac{(\bar{X} - \mu)}{S_{\bar{X}}}$$

$\bar{X}$ = SAMPLE MEAN

μ = POPULATION MEAN

$S_{\bar{X}}$ = STANDARD ERROR OF MEAN

WAS THEN USED TO ENTER A TABLE OF T VALUES AND OBTAIN THE PROPER PROBABILITY. FOR THESE CALCULATIONS, μ WAS ASSUMED TO BE THE THEORETICAL B VALUE.

CHAPTER VI

RESULTS

Preliminary Tests

By use of the deflection data from the preliminary tests, the EI value was found to be 318.58×10^6 in^2-lb. Using the strain data and an assumed value of E, 29.5×10^6 lb/in^2, an EI value of 337.31×10^6 lb-in^2 was obtained. This value was approximately six per cent larger than the value determined by deflection readings. For calculation purposes, the EI value obtained by use of the deflection data was used.

In order to compare the experimental results with other available data, the I value was needed which was found by dividing the EI value obtained from deflection data by an estimated value of E, 29.5×10^6 lb/in^2. This value of I, 10.8 in^4 was approximately 20 per cent less than the I value given in the _Light Gage, Cold-Formed Steel Design Manual_. Two reasons for this low value are suggested. First, measurements of the sections indicated a thickness of approximately 0.104 in, and the tabulated value of I was for a thickness of 0.105 in. The second factor was the lateral instability or twisting of the sections due to inadequate fastenings holding the C-shaped sections together. The _Light Gage, Cold-Formed Steel Design Manual_ specifies the maximum permissible longitudinal spacing of the fastenings for flexural members as L/6 where L is the span of the member. This would require a total of five fastenings between supports for the frame member, and only four were used for these experiments. The reason for using only four fastenings was to represent the prototype frame, as nearly as possible, which had spaces between fastenings up to 93 inches in length. Observations during the test at high loads showed that twisting of the member sections was very apparent.

Bending moments found by electrical resistance strain gage measurements and the I value determined by use of the deflection data were found to be within 7 per cent of the theoretically determined bending moments. It was also found that strain measurements indicated movement of the centroid away from the neutral axis. For two members, this movement was toward the tension side of the member, and for the other two members it was toward the compressive side. This movement was thought to be attributed to placing the strain gages on both sections of the member, but on opposite sides as shown in Figure 13. A slightly different EI value for one section due to material or manufacturing differences could account for this effect. To avoid this effect in the prototype frames, the strain gages were mounted on either side of one section of the member for all subsequent tests.

Statistical analysis of the deflection data indicated a large amount of variation due to differences among frame members. For the strain data, the largest amount of variation was due to the position of the strain gages. This emphasizes the importance of correctly positioning the strain gages. The analysis of variance tables for the deflection and strain gage readings are listed in Appendix C.

FRAME TESTS

DEFLECTION RESULTS

RESULTS FROM THE GRAVITY LOADING TESTS ON BOTH THE HINGELESS AND TWO-HINGED FRAMES INDICATED THAT THE FRAMES WERE, IN MOST CASES, CONSIDERABLY MORE FLEXIBLE THAN CALCULATED WITH THE EXPERIMENTALLY DETERMINED EI VALUE. THE MEASURED PEAK DEFLECTION OF THE HINGELESS FRAMES WAS 33.45 PER CENT GREATER THAN THE CALCULATED VALUE AND 13.85 PER CENT GREATER FOR THE TWO-HINGED FRAME. AT THE HAUNCH THE DEFLECTION WAS 21.40 PER CENT GREATER THAN CALCULATED FOR HINGELESS FRAMES AND 1.84 PER CENT GREATER FOR THE TWO-HINGED FRAME.

POSSIBLE EXPLANATIONS FOR THE DIFFERENCES BETWEEN THE THEORETICAL AND EXPERIMENTAL DEFLECTION VALUES ARE BASED ON THE ASSUMPTIONS MADE IN THE THEORETICAL ANALYSIS. THE FIRST ASSUMPTION THAT THE MATERIAL BEHAVES ELASTICALLY AND OBEYS HOOKE'S LAW APPEARED TO BE IN GOOD AGREEMENT WITH THE EXPERIMENTAL BEHAVIOR SINCE THE DEFLECTION OF THE FRAME VERSUS LOAD OF THE FRAME APPEARED TO BE A LINEAR RELATIONSHIP. THIS WAS REVEALED BY THE HIGH CORRELATION COEFFICIENTS FOR DEFLECTION IN INCHES VERSUS TOTAL APPLIED LOAD IN POUNDS. FOR SUCH CYCLES OF LOADING, THESE CORRELATION COEFFICIENTS WERE ALL ABOVE 0.924, AND 83 PER CENT OF THEM WERE ABOVE 0.990. DEVIATIONS OF THE EXPERIMENTAL POINTS FROM A STRAIGHT LINE APPEARED TO BE RANDOM OVER THE ENTIRE REGRESSION LINE AS CAN BE OBSERVED IN FIGURE 25. THESE HIGH CORRELATION COEFFICIENTS ALSO HELP RULE OUT THE POSSIBILITY OF SLIPPAGE AT THE BOLTED JOINTS. SLIPPAGE WOULD PROBABLY HAVE BEEN INITIATED AT SOME PARTICULAR LOAD AND WOULD BE EVIDENCED BY AN ABRUBT CHANGE IN SLOPE OF THE REGRESSION LINE CAUSING POOR CORRELATION.

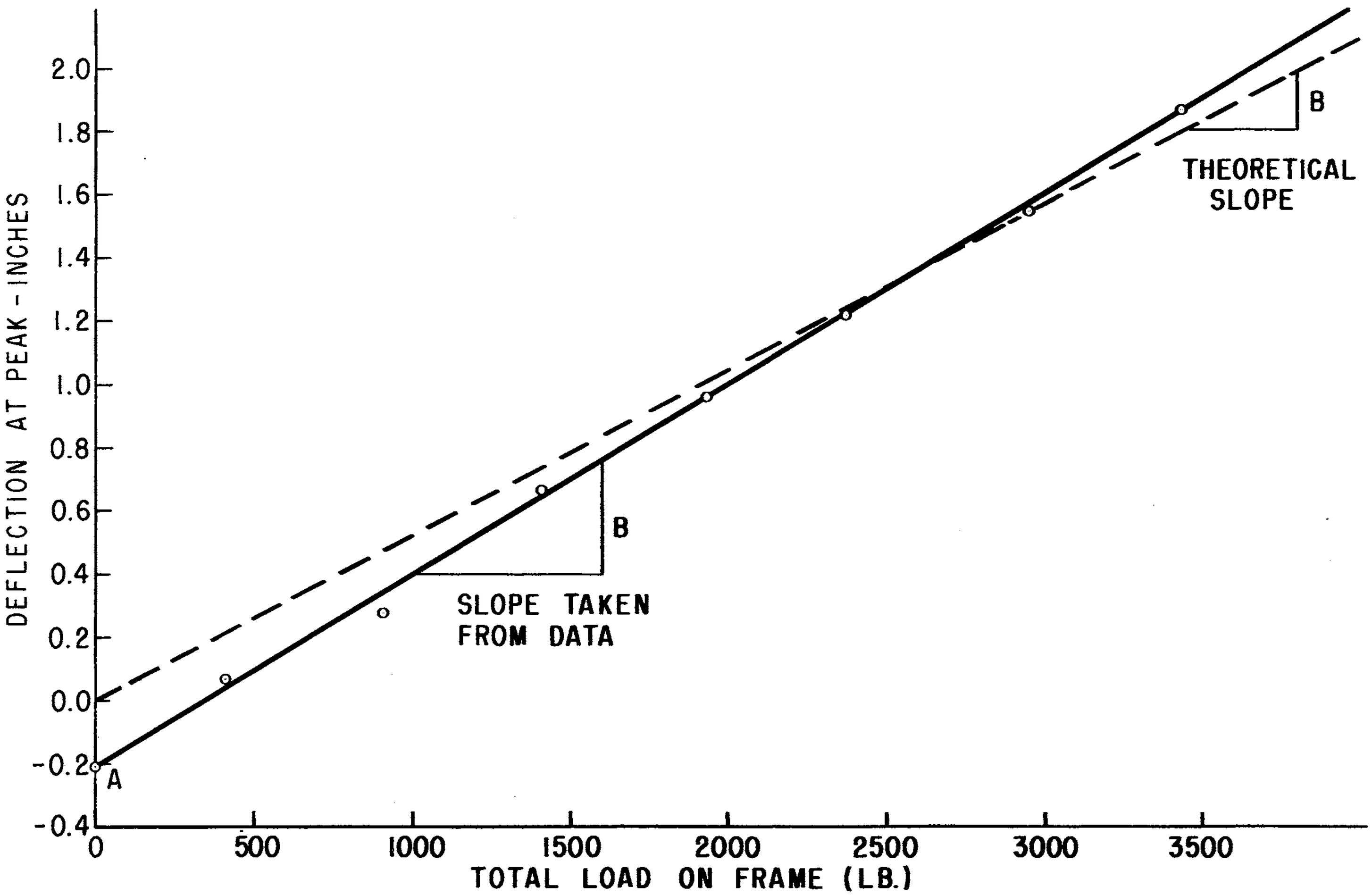


FIGURE 25. DEFLECTION AT PEAK VERSUS LOADING OF FRAME.

The next assumption was that deformations due to shearing forces are small. Shear strain would add little to the frame deformation since the frame members were long compared to their cross-section dimensions. Shear strain could account for some of the calculated deflection deviation between experimental and calculated results in the hingeless frame because of the larger perpendicular shearing forces, as compared to the two-hinged frame in the shear diagrams of Figures 7, 8, and 9.

The next possible cause for the large deflections was the twisting of the member sections. This twisting was very noticeable at high gravity loads and at portions of the frame which experienced high bending moments. The portions of the frame between the haunch and the fastenings approximately one-half the distance between haunch and peak appeared to have the largest amount of twisting. At this position, the sections twisted from their original spacing of 3/16 inch to approximately 1/2 inch or more on the tension side of the member while the spacing was completely closed on the compression side. To test the effect of additional fastenings between the frame sections, four extra fastenings were installed in one two-hinged frame. These fastenings were placed 1/3 of the distance between original fastenings inward from both ends of the frame top members. Gravity loading tests of the frame with these extra fastenings indicated deflection at the peak to be 0.67 per cent greater than calculated and 9.67 per cent less at the haunch. From these results, it would appear that most of the difference between theoretical and experimental deflections could be accounted for by the twisting or lateral instability of the member sections. Only two loading cycles were run

ON ONE FRAME FOR THIS EXPERIMENT ON FASTENER SPACING. HOWEVER, APPROXIMATELY THE SAME VALUES WERE OBTAINED FOR BOTH CYCLES.

THIS SAME EFFECT WOULD LIKELY ACCOUNT FOR THE LARGER PERCENTAGE DIFFERENCES BETWEEN THE THEORETICAL AND EXPERIMENTAL DEFLECTION ON VALUES OF HINGELESS FRAMES AS COMPARED TO THE TWO-HINGED FRAMES. A LARGE PORTION OF THIS COULD POSSIBLY BE ATTRIBUTED TO THE LOWER PORTION OF EACH COLUMN MEMBER. THIS PORTION HAD A SPAN OF APPROXIMATELY 77 INCHES BETWEEN FASTENINGS, AND EXPERIENCED HIGH BENDING MOMENTS UNDER GRAVITY LOADS, FIGURE 7. FOR THE TWO-HINGED FRAME, THIS PORTION DEVELOPED COMPARATIVELY SMALL BENDING MOMENTS AS THE BENDING MOMENTS APPROACHED ZERO AT THE LOWER END OF THIS MEMBER.

SUPPORT ROTATION OF THE HINGELESS FRAMES PRODUCED 27.51 PER CENT LESS DEFLECTION AT THE PEAK AND 32.18 PER CENT LESS DEFLECTION AT THE HAUNCH THAN THE CALCULATED VALUES. SINCE THE EXPERIMENTAL VALUES FOR BOTH DEFLECTIONS AND BENDING MOMENTS, AS DETERMINED BY STRAIN, WERE LESS THAN THE CALCULATED VALUES IN ALL CASES, ROTATION OF THE SUPPORTS MAY NOT HAVE BEEN AS LARGE AS MEASURED. THIS COULD BE DUE TO THE SUPPORT BENDING BETWEEN THE POINTS OF ROTATION MEASUREMENT. HOWEVER, IT DOES NOT SEEM LIKELY THAT SUPPORT BENDING WOULD CAUSE DIFFERENCES AS GREAT AS THOSE OBTAINED SINCE THE SUPPORTS WERE OVER SEVEN TIMES AS STIFF AS THE FRAME MEMBER.

ANOTHER POSSIBLE CAUSE FOR THE LOW DEFLECTION AND BENDING MOMENT VALUES IS LOCAL BUCKLING OF THE FRAME MEMBERS JUST ABOVE THE FRAME SUPPORTS. THIS COULD BE CAUSED BY THE HIGH BENDING MOMENTS AT THIS POSITION WHICH WERE APPROXIMATELY FOUR TIMES GREATER THAN AT ANY OTHER POSITION IN THE FRAME.

For translational movement of the hingeless frame supports, the deflection at the peak was 28.06 per cent greater than expected and 18.52 per cent less at the haunch than expected. Since the deflection at the peak was greater than calculated, it indicates that full support movement was likely obtained. However, the deflection of the support was probably of the same magnitude as for the support rotation experiments since approximately the same bending moments were generated at the supports.

The deflection due to lateral support movement of the two-hinged frame was 26.07 per cent less than calculated for deflection at the peak, and 0.53 per cent greater at the haunch.

A large portion of the difference between the experimental and theoretical results for the support movement may be due to friction between the support brackets and the frame. Friction due to the weight of the frame resting on the brackets would be constant and, therefore, have the same effect for each increment of loading or support movement. However, if the friction was great enough so that several loading or support movement increments were required to overcome it, the results could be effected appreciably. The tests conducted on the first frame were made using a loading and unloading portion for each loading cycle. Results from these tests indicated two regression lines with approximately the same slope but considerable distance between intercepts. It was noted also that one or possibly two unloading increments were applied before the data again followed a straight line representing deflection versus load. These results were thought to be attributed to the friction acting in the entire testing system.

To eliminate as much friction error as possible, only loading cycles were used for all subsequent tests.

To test the effect of end restraint from the brackets for attaching the two-hinged frame to a foundation, six loading cycle tests on three different frames were made. The results obtained were 4.03 per cent less deflection at the peak and 4.74 per cent less deflection at the haunch as compared to the deflections for the pinned-end condition. These values are applicable only for the type brackets used in this test, however, the brackets used would probably be a typical representation of other types used. This difference was quite small compared to other errors. Therefore, the assumption of pinned-end conditions is valid for these tests. Statistical analysis comparing the deflections of the pinned and fixed-end supports showed high variance ratios as can be noted in Appendix C.

In regard to slippage of the bolted joints at the haunches, it was found in all cases that some initial joint movement occurred during the preloading cycles. This average rotation due to slippage between the top member of the frame and the haunch gusset plate was approximately 0.0104 radians measured at the center of the bolt spacings. The rotation of the column with respect to the gusset plate was found to be very small and negligible in most cases. It was found that properly torquing the bolts connecting the frame members and gusset plates after the preloading cycles eliminated slippage during the loading tests. In actual construction, loosening of the bolts during initial loading could have an appreciable effect on the structure. It is noted that frame parts were made in an engineering

LABORATORY WHICH DID NOT HAVE PRECISION EQUIPMENT FOR LOCATING THE BOLT HOLES AS WOULD BE USED IN PLANT FABRICATION. THIS MISALIGNMENT COULD POSSIBLY ACCOUNT FOR MOST OF THE SLIPPAGE THAT OCCURED.

IT WAS NOTED WHEN CONDUCTING THESE TESTS THAT THE DEFLECTION AT THE PEAK CONTINUED TO INCREASE WITH TIME UNDER A SUSTAINED MAXIMUM LOAD. TO INVESTIGATE THE EFFECT OF TIME ON THE DEFLECTION AT THE PEAK, A TWO-HINGED FRAME WAS LOADED AND DEFLECTION READINGS TAKEN AT VARIOUS TIME INTERVALS. THE TOTAL LOAD APPLIED TO THE FRAME WAS 3,317 LB. THE RESULTS FROM THESE TESTS CAN BE OBSERVED IN FIGURE 26. THE TOTAL DEFLECTION INCREASE DURING 868 MINUTES WAS 0.307 IN. WHEN THE LOAD WAS RELEASED, THE PEAK DEFLECTION, AFTER 48 HOURS, HAD RETURNED TO WITHIN 0.130 IN OF THE ORIGINAL STARTING VALUE, INDICATING THAT 42 PER CENT OF THE TIME-DEPENDENT DEFLECTION WAS PERMANENT SET. THIS LAG IN DEFLECTION WITH TIME COULD POSSIBLY BE BEST EXPLAINED BY HYSTERESIS EFFECTS, THE LAGGING OF A PHYSICAL EFFECT ON A BODY BEHIND ITS CAUSE. IN THE CASE OF AN ELASTIC MATERIAL AS USED IN THESE TESTS, THE HYSTERESIS EFFECT WOULD BE THE ELASTIC AFTER EFFECT WHICH IS DUE TO THE THERMOELASTIC PROPERTIES OF THE MATERIAL.

THE DEFLECTION WHICH TOOK PLACE IN 868 MINUTES, WHICH AMOUNTS TO 15 PER CENT OF THE TOTAL DEFLECTION, COULD HAVE A SIGNIFICANT EFFECT ON THE RESULTS FROM PROLONGED TESTS. FOR THE PRESENT TESTS, THE TIME FOR EACH LOADING CYCLE WAS APPROXIMATELY 45 MINUTES; THEREFORE, THE TIME EFFECT WAS SMALL. IF THE RESULTS OBTAINED FROM THESE EXPERIMENTS WERE TO BE APPLIED TO LONG-TERM LOADING CONDITIONS, THEY SHOULD BE CORRECTED FOR TIME-DEPENDENT DEFLECTION.

COMPARISON OF THE DEFLECTIONS OF THE HINGELESS AND TWO-HINGED FRAMES, REVEALED THAT THE HINGELESS FRAME HAD 24 PER CENT LESS

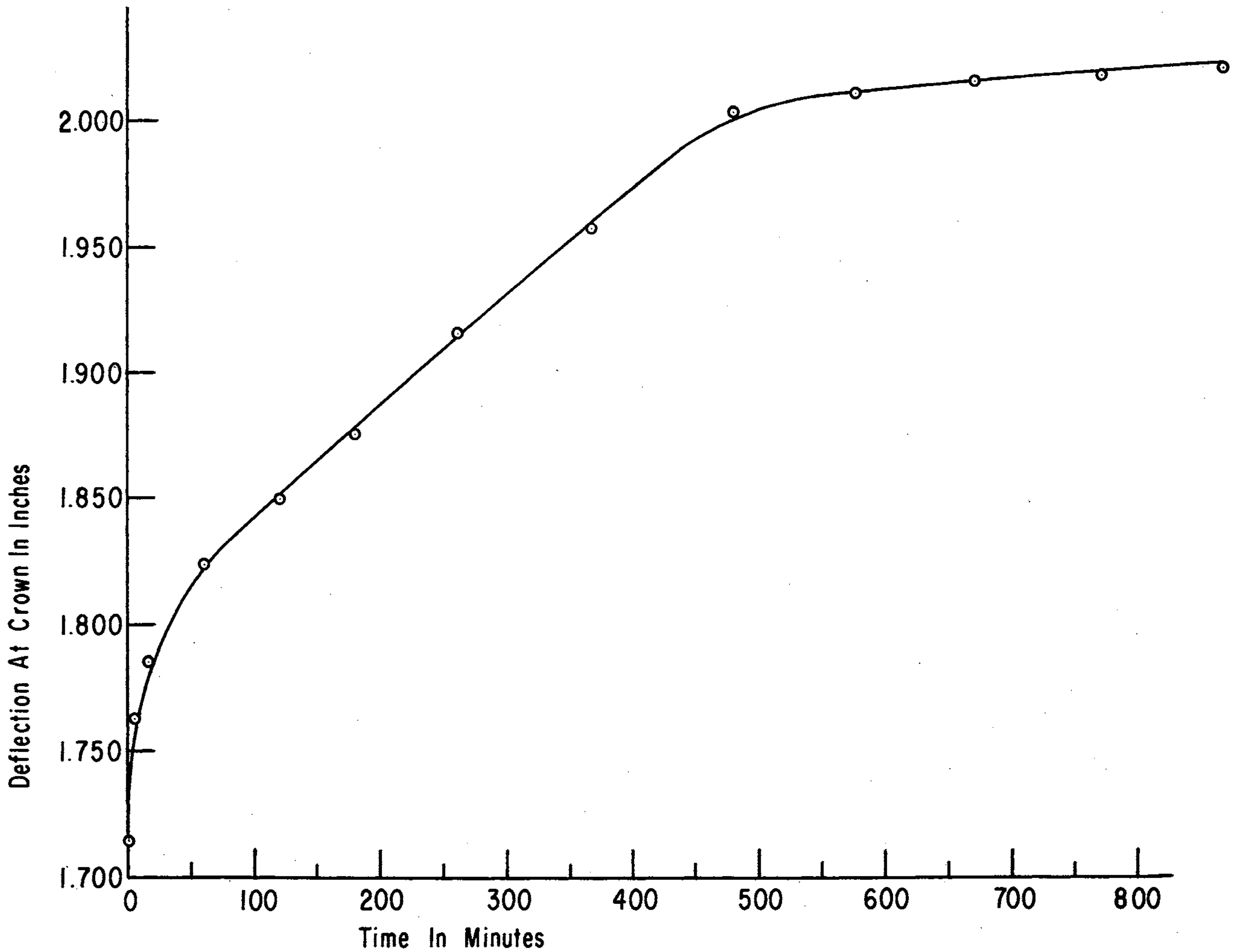


FIGURE 26. TIME VERSUS DEFLECTION AT PEAK FOR TWO-HINGED FRAME.

deflection at the peak and 23 per cent less deflection at the haunch as compared to deflection of the two-hinged frame. These experimental results are considerably less than the theoretical difference of 35 per cent for both positions.

Analysis of variance for both the peak deflection and haunch data showed high variance ratios due to differences between the end conditions. This analysis of variance for the haunch deflection data showed high variance ratios due to both loading cycles and interaction between loading cycles and end conditions. To determine the variance ratios due to the hingeless and two-hinged frames, a randomized complete-block design analysis of variance was used as presented by Steel and Torrie (22). For this analysis, each cycle was treated as a complete randomized block. The variance ratios due to differences between frames had a significance level above 90 per cent in all but one case.

Strain Results

The results from the strain readings were used to calculate both axial loads and bending moments. As can be observed in Figure 24, the amount of strain due to axial loading was very small as compared to the strain due to bending. Therefore, a small error in either of the two readings used to make these calculations could cause considerable error in computation of the axial load of the member. This was found to be the case in most instances since the difference between the experimental and theoretical values ranged from 86.61 per cent greater to 86.71 per cent less than the theoretical values. Another factor is the different behaviors of

THE SECTION FLANGES IN COMPRESSION AND TENSION. ALSO, REDISTRIBUTION OF THE STRESSES DUE TO TWISTING AND LATERAL INSTABILITY OF THE MEMBER SECTIONS POSSIBLY HAD SOME EFFECT. OTHER FACTORS BELIEVED TO CONTRIBUTE TO THESE ERRORS WERE FRICTION OF THE FRAME SLIDING ON THE SUPPORT BRACKETS AND VARIATION OF TEMPERATURE DURING TESTING WHICH COULD NOT BE CONTROLLED. THE DATA FOR AXIAL LOAD ARE LISTED IN APPENDIX B. ALSO, A PLOT OF AXIAL LOADING VERSUS LOADING OF THE FRAME IS SHOWN IN FIGURE 27. THIS DATA WERE FROM A GRAVITY-LOADING CYCLE WHICH HAD AN EXCEPTIONALLY HIGH CORRELATION COEFFICIENT.

RELATIVELY HIGH LINEAR CORRELATION COEFFICIENTS WERE OBTAINED FOR BENDING MOMENTS IN FT-LB VERSUS LOAD AND SUPPORT MOVEMENTS, RESPECTIVELY OF THE FRAMES. FOR APPROXIMATELY 75 PER CENT OF THE LOADING CYCLES, THESE COEFFICIENTS WERE ABOVE 0.99, WHICH INDICATES A GOOD FIT OF THE DATA TO THE REGRESSION LINE. THIS CAN BE OBSERVED IN FIGURE 28.

FOR THE HINGELESS FRAME WITH GRAVITY LOADING, THERE WAS 31.63 PER CENT MORE BENDING MOMENT DEVELOPED AT POSITION 44 NEAR THE PEAK THAN EXPECTED. THIS HIGH VALUE OF BENDING MOMENTS CORRESPONDS TO A HIGH VALUE OF DEFLECTION FOR THE SAME TESTING CONDITIONS. A POSSIBLE CAUSE FOR THESE HIGH VALUES COULD BE EITHER A SMALL AMOUNT OF SLIPPAGE OR MORE ELASTIC DEFORMATION AT THE HAUNCH JOINT PLATES THAN IN THE FRAME MEMBERS. SLIPPAGE AT THE HAUNCH JOINT WAS NOT DETECTED BY THE ROTATION MEASURING EQUIPMENT EXCEPT IN TWO OR THREE CASES IN WHICH IT WAS SMALL. HOWEVER, DURING HIGH GRAVITY LOADS ROTATION OF THE TOP FRAME MEMBER WAS APPROXIMATELY 0.0017 RADIANS MEASURED AT THE CENTER OF THE BOLT GROUP CONNECTING THE TOP MEMBER AND GUSSET PLATE. THIS ROTATION RETURNED BACK TO ZERO WHEN THE LOAD WAS RELEASED.

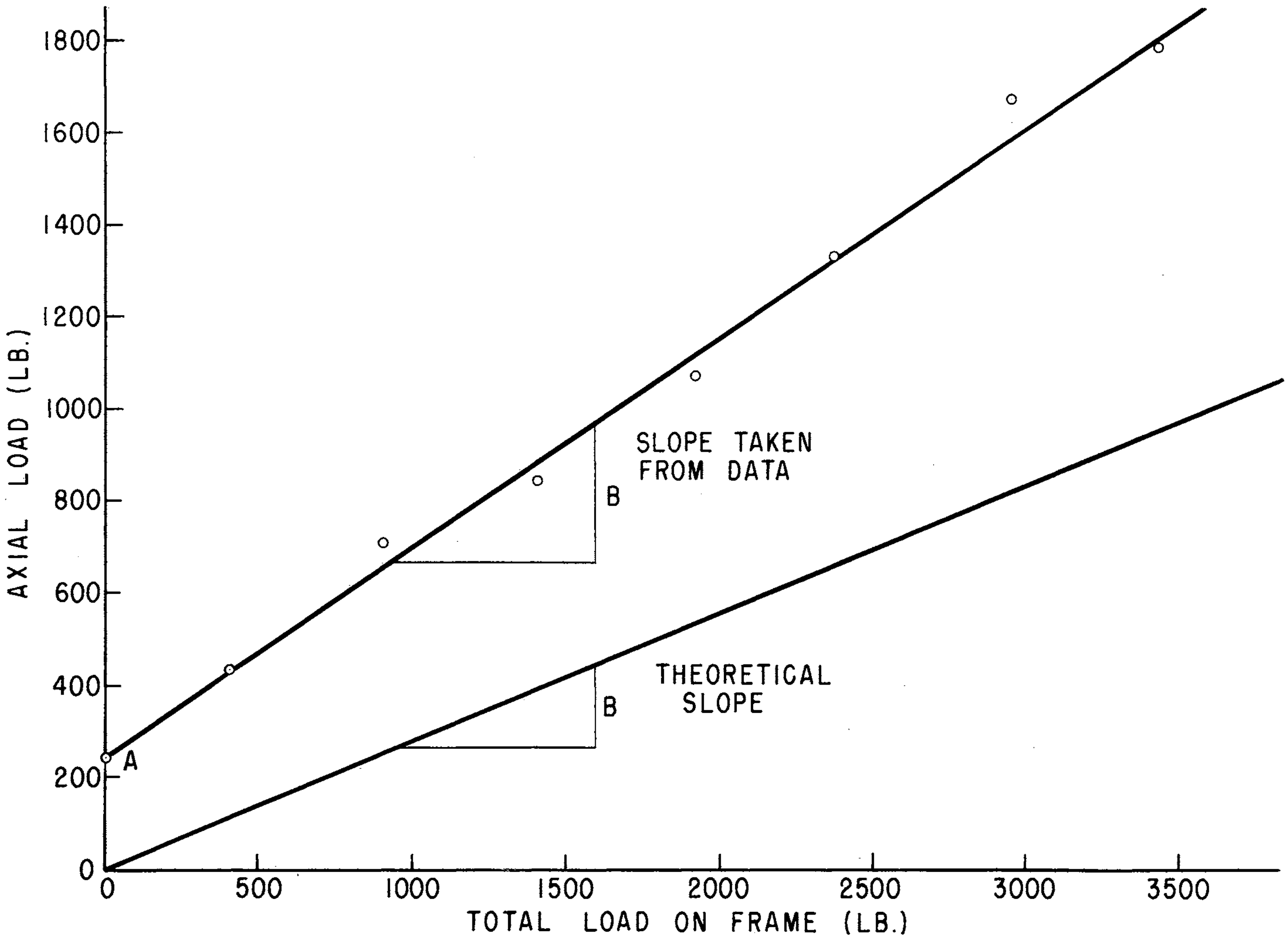


FIGURE 27. AXIAL LOAD VERSUS LOADING OF FRAME.

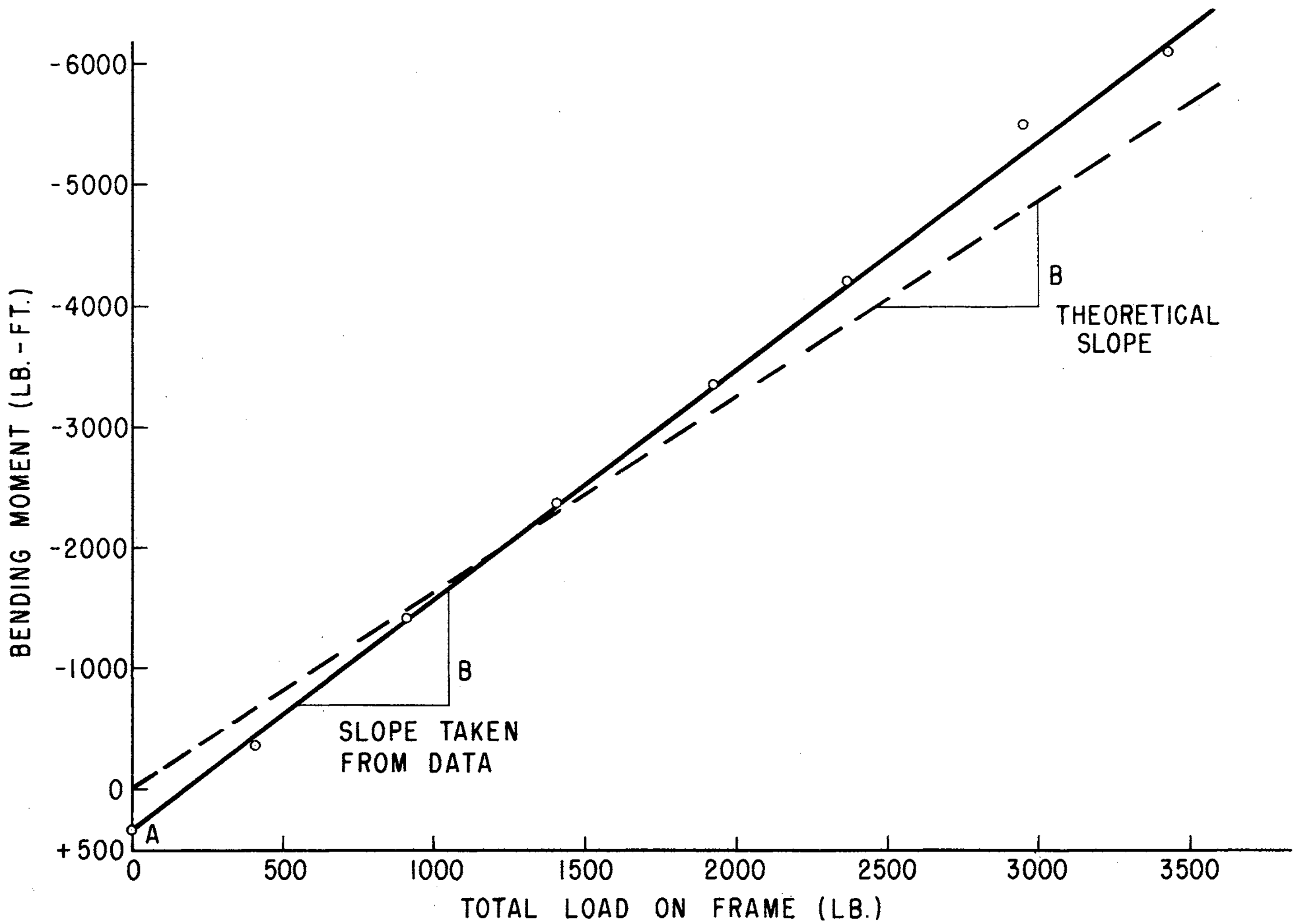


FIGURE 28. BENDING MOMENT VERSUS LOADING OF FRAME.

Therefore, it was thought to be elastic and could have possibly contributed to the high bending moments at position 44. Bending moments at positions 22 and 33 near the haunch were 5.42 per cent and 14.96 per cent larger than expected. For position 11, the bending moments were 27.71 per cent smaller than expected. This was possibly due to the support end of the frame rotating or deflecting between rotation measuring dial micrometers. A small amount of rotation here would have decreased the bending moments at position 11 and increased the bending moments at positions 22, 33, and 44. This is in conformity with the experimental results. Another fact which helps to verify this is that for the two-hinged frame under gravity load, bending moments were developed which were close to calculated values as shown in Appendix B. Therefore, if the bending moments can be predicted for one frame type, they should be predictable for other frame types also unless some assumption used in the theoretical analysis was incorrect.

For the two-hinged frame, the end conditions were assumed to be pinned. This condition was not difficult to achieve in the laboratory. However, the hingeless frame supports were assumed to be perfectly rigid for the theoretical analysis. This condition is harder to achieve since only a small amount of movement can cause considerable effect on the frame stresses. Using data for this experiment, it was found that the bending moment at position 11 was 0.42 ft-lb/lb of applied load less than expected. From the support rotation experiment, it was found 407,112 ft-lb of bending moments were produced per radian of support rotation. Therefore, only 1.03 $\times 10^{-6}$ radians of support movement could account for the error in the

BENDING MOMENT AT POSITION II DUE TO ONE POUND OF LOAD APPLIED TO THE FRAME. THIS WOULD MEAN A MOVEMENT OF 49.44×10^{-6} IN OF MOVEMENT AT THE BOTTOM OF THE FRAME SUPPORT ASSUMING THE TOP OF THE FRAME SUPPORT REMAINED RIGID. IF A LOAD OF 3,000 POUNDS WAS APPLIED TO THE FRAME, 0.148 INCHES OF MOVEMENT AT ONE SUPPORT WOULD ACCOUNT FOR THE ERROR. IT IS NOT LIKELY THAT THIS MUCH ERROR COULD HAVE OCCURRED SINCE THE DIAL MICROMETERS USED TO MEASURE SUPPORT MOVEMENT COULD BE READ ACCURATELY TO ONE THOUSANDTH OF AN INCH AND THE SUPPORTS WERE ADJUSTED BY THESE DIAL MICROMETERS AFTER EACH LOADING INCREMENT. ALTHOUGH THE SUPPORT MOMENT WOULD NOT LIKELY ACCOUNT FOR ALL THE ERRORS BETWEEN THEORETICAL AND EXPERIMENTAL RESULTS, IT APPEARS TO ACCOUNT FOR A PORTION OF IT. THE BENDING MOMENT RESULTS OF THE SUPPORT MOVEMENT EXPERIMENTS INDICATED LARGE VARIANCES FROM THE THEORETICAL VALUES IN NEARLY ALL CASES. THESE DIFFERENCES, AS IN THE PREVIOUS CASES, WERE ATTRIBUTED TO ERRORS IN SUPPORT ROTATION MEASUREMENT AND FRICTION OF THE FRAME SLIDING ON THE SUPPORT BRACKETS.

OBSERVATIONS OF STRAIN MEASURED IN MICROINCHES PLOTTED AGAINST TOTAL LOAD APPLIED TO THE FRAME IN POUNDS INDICATED A DEVIATION FROM A STRAIGHT LINE AT APPROXIMATELY 400 TO 450 MICROINCHES OF STRAIN. THIS EFFECT SHOWN IN FIGURE 29 WAS NOTED IN ALL OF SEVEN DIFFERENT STRAIN VERSUS LOADING DIAGRAMS PLOTTED. IT COULD BE EXPLAINED BY BUCKLING EFFECTS. IT DOES NOT APPEAR TO BE DUE TO LOCAL BUCKLING, HOWEVER, SINCE THE σ_{cr} CRITICAL BUCKLING STRESS GIVEN AS (23)

$$\sigma_{cr} = K \frac{\pi^2 E}{12 \left(1 - \mu^2\right) (W/T)^2}$$

WHERE K = COEFFICIENT DEPENDING ON END SUPPORTS

E = MODULUS OF ELASTICITY

μ = POISSION'S RATIO

T = PLATE THICKNESS

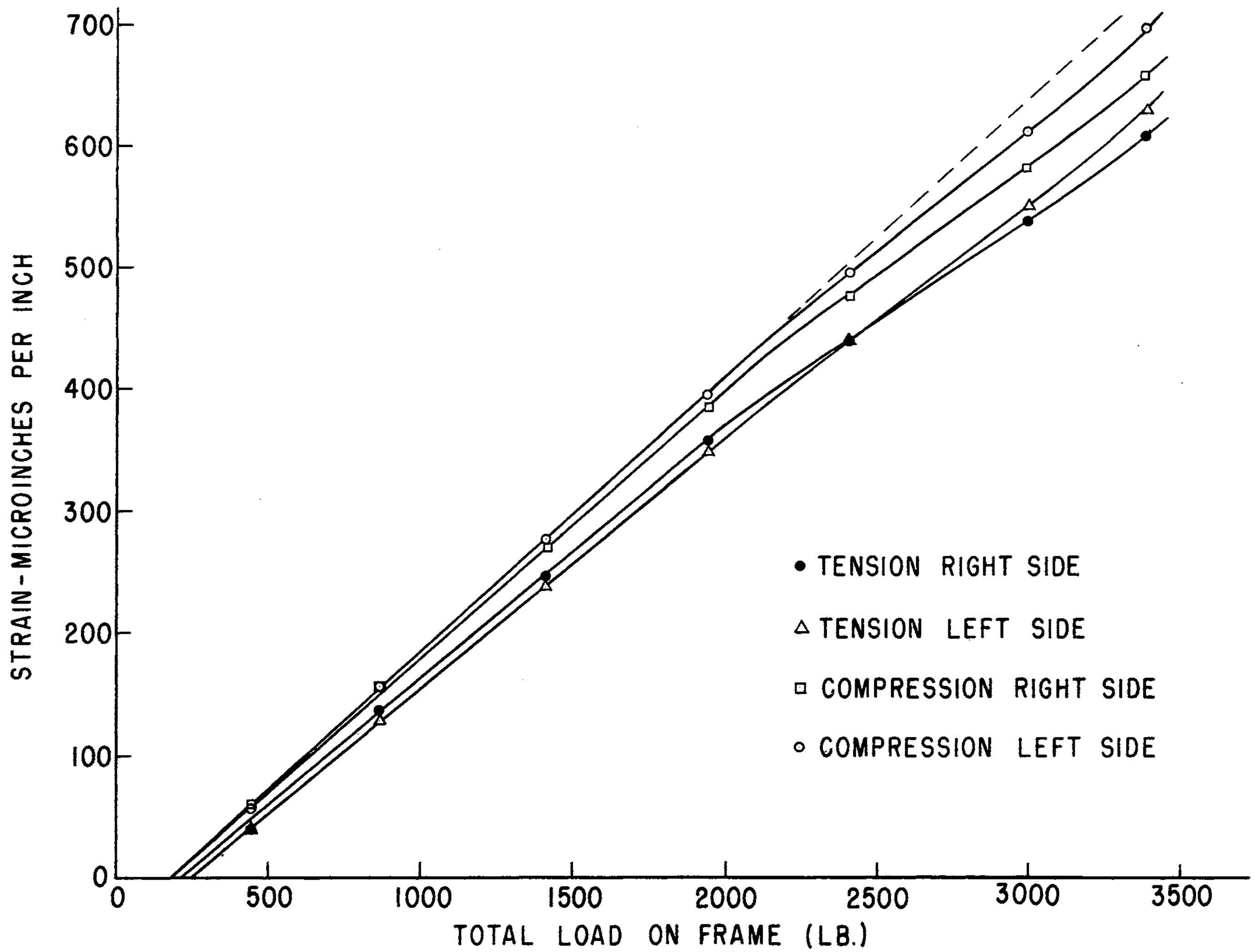


FIGURE 29. STRAIN VERSUS LOADING OF FRAME.

W = WIDTH OF PLATE

GIVES A VALUE MUCH HIGHER THAN THE VALUE AT WHICH THE DEVIATIONS OCCURRED IN THE EXPERIMENTAL RESULTS. ANOTHER POSSIBLE CAUSE COULD HAVE BEEN PLASTIC DEFORMATION AT SOME POINT IN THE FRAME WHICH WOULD RELIEVE THE STRESSES IN OTHER PORTIONS OF THE FRAME. THIS VERY LIKELY COULD HAVE OCCURRED SINCE PLASTIC DEFORMATION WAS OBSERVED IN THE TIME VERSUS DEFLECTION TESTS.

COMPARING THE BENDING MOMENTS DEVELOPED IN THE TWO-HINGED AND HINGELESS FRAMES, THE FOLLOWING RESULTS WERE FOUND. FOR POSITION 22, EXPERIMENTAL RESULTS SHOWED 19.05 PER CENT LESS BENDING MOMENT IN THE HINGELESS FRAME AS COMPARED TO 25.54 PER CENT LESS FOR THEORETICAL RESULTS. AT POSITION 33, THE HINGELESS FRAME HAD 11.27 PER CENT LESS AS COMPARED TO 10.68 PER CENT FOR THEORETICAL RESULTS; AND POSITION 44 SHOWED 22.02 PER CENT LESS FOR THE HINGELESS FRAME AS COMPARED TO 36.13 PER CENT FOR THEORETICAL RESULTS. ALL THE ABOVE PERCENTAGES ARE BASED ON RESULTS FROM THE TWO-HINGED FRAME. ALTHOUGH THE EXPERIMENTAL RESULTS SHOWED LESS DIFFERENCE AT POSITIONS 22 AND 44 THAN THEORETICALLY CALCULATED, THESE DIFFERENCES OF 19.05 PER CENT AND 22.02 PER CENT RESPECTIVELY COULD HAVE CONSIDERABLE INFLUENCE IN FRAME DESIGNS. AT POSITION 33, THE BENDING MOMENTS IN BOTH THE HINGELESS AND TWO-HINGED FRAMES WERE HIGHER THAN AT ANY OTHER POINT MEASURED AND ALSO WERE HIGHER THAN EXPECTED IN BOTH CASES.

STATISTICAL ANALYSIS OF VARIANCE AS PRESENTED IN THE ANALYSIS OF VARIANCE TABLES IN APPENDIX C SHOW LARGE VARIANCE RATIOS DUE TO END CONDITIONS AS COMPARED TO RANDOM ERRORS. THIS INDICATED DEFINITE DIFFERENCES BETWEEN BENDING MOMENTS AT POSITIONS 22, 33, AND 44 IN THE TWO-HINGED AND HINGELESS FRAMES.

Position II was not considered since the difference between the bending moments in the hingeless and two-hinged frames was so great. Analysis of variance to determine variance ratios due to frames in the hingeless and two-hinged frames was obtained in the same manner as for the deflection data. The ratios of variance due to the differences in frames were all found to have levels of significance above 62.7.

CHAPTER VII

SUMMARY AND CONCLUSIONS

Summary

This study was conducted to evaluate the stiffness of geometrically similar hingeless and two-hinged light gage, cold-formed steel frames. Also the secondary stresses developed due to support movement were investigated. This was done experimentally for both cases. The analytical results were obtained by use of the moment-area method. To compare the stiffness of the geometrically similar hingeless and two-hinged frames, four frames of each type were tested. These tests were conducted simulating gravity loads which were applied hydraulically. To study the effect of support movement, rotational support movement and translational support movement of the hingeless frame and lateral support movement of the two-hinged frames were studied.

Conclusions

The conclusions drawn from this study are as follows:

1. The EI value of the frame members was found to be 19.4 per cent less than the expected value, which was based on the calculated value.

2. Peak deflections for the hingeless and two-hinged frames were found to be 33.45 per cent and 21.42 per cent respectively

GREATER THAN THE EXPECTED VALUES FOR WHICH THE PERCENTAGES WERE BASED. THE APPARENT REASON FOR THE LACK IN STIFFNESS WAS TWISTING OF THE FRAME MEMBER SECTIONS DUE TO INADEQUATE FASTENINGS WHICH HELD THE TWO C-SHAPED SECTIONS TOGETHER.

3. THE USE OF FOUR EXTRA FASTENINGS, WHICH CONSISTED OF A 3/16 IN PLATE BOLTED BETWEEN THE TWO C-SHAPED SECTIONS MAKING UP THE FRAME MEMBERS WITH TWO 1/2 IN BOLTS SPACED 4 IN APART PERPENDICULAR TO THE FRAME MEMBER, WAS FOUND TO REDUCE THE DEFLECTION AT THE PEAK FOR A TWO-HINGED FRAME 10.18 PER CENT.

4. RESTRAINING THE TWO-HINGED SUPPORTS WAS FOUND TO REDUCE DEFLECTION AT THE PEAK 4.03 PER CENT AS COMPARED TO THE PEAK DEFLECTION FOR THE PINNED-END SUPPORT CONDITION.

5. DEFLECTIONS FOR THE HINGELESS FRAMES WERE FOUND TO BE 24 PER CENT LESS AT THE PEAK AND 23 PER CENT LESS AT THE HAUNCH AS COMPARED TO DEFLECTIONS FOR THE TWO-HINGED FRAMES. ANALYTICAL CALCULATIONS INDICATED 35 PER CENT LESS DEFLECTION AT BOTH THE PEAK AND HAUNCH AS COMPARED TO THE TWO-HINGED FRAME DEFLECTION VALUES.

6. DEFLECTIONS AT THE PEAK FOR THE HINGELESS FRAMES, DUE TO SUPPORT MOVEMENTS, WERE 27.51 PER CENT LESS FOR SUPPORT ROTATION AND 28.06 PER CENT GREATER FOR TRANSLATIONAL SUPPORT MOVEMENT AS COMPARED TO THE THEORETICAL VALUES. FOR LATERAL SUPPORT MOVEMENT OF THE TWO-HINGED FRAMES, THE DEFLECTION AT THE PEAK WAS 26.07 PER CENT LESS AS COMPARED TO THE THEORETICAL VALUE.

7. AXIAL LOADS IN THE FRAME MEMBERS AS DETERMINED BY ELECTRICAL RESISTANCE STRAIN GAGES WERE FOUND TO VARY FROM 86.70 PER CENT GREATER TO 86.71 PER CENT SMALLER THAN THEORETICAL VALUES.

8. Bending moments determined by use of electrical resistance strain gages were found to be within 7 per cent of the values determined theoretically.

9. Bending moments for the hingeless frames 6 in above the supports were 27.70 per cent less as compared to the theoretical value for gravity loading indicating possible yielding of the supports.

10. Highest bending moments due to gravity loads were encountered at position 33, the position just above the haunch joint, in both the hingeless and two-hinged frames.

11. Experimentally determined bending moments corresponded much more closely to theoretical values for the two-hinged frame with the largest difference being 15.73 per cent greater as compared to the largest difference of 31.63 per cent for the hingeless frame.

12. Bending moments due to support movements were found to vary considerably from theoretical values with the differences between experimental and theoretical values ranging from 42.48 per cent larger to 32.86 per cent less as compared to theoretical values. Friction between the frames and support brackets and yielding of the supports were believed to be responsible for large portions of these differences.

13. Assuming the supports of a hingeless frame to yield outward at the ground level 1/8 in and to rotate about a point 2/3 of the total depth of the support, the following bending moments from experimental results would be produced: position 11, (-1,836 ft-lb), position 22, (-456 ft-lb) position 33, (-132 ft-lb) and position 44, (+678. ft-lb). The effect these bending moments would have on a frame gravity loaded by a 3,000 lb load are: positon 11, 56 per cent

less; position 22, 10 per cent greater; position 33, 2.6 per cent greater and position 44, 21 per cent greater. Therefore, when designing hingeless frames, possible support yielding should be taken into consideration.

14. Analytical analysis showed bending moments in the two-hinged frames to be greater for all loading conditons excepting grain load and grain load plus snow load loading conditions.

Suggestions for Further Studies

1. A more detailed study should be made of the effects on stiffness of the frames due to extra fastenings between the frame member sections.

2. A study should also be made with the frames erected outdoors as they would be in actual construction. This could also include erection of the hingeless frames in two or more varied soil types to study the effect of soil resistance and support movement on the stiffness of the frames.

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APPENDIX A

FORTRAN PROGRAMS USED TO ANALYZE EXPERIMENTAL DATA

TABLE A.I

FORTRAN PROGRAM USED TO ANALYZE STRAIN AND DEFLECTION DATA

```
C 0000 0 PROJECT 633
C 0000 0 RESULTS FROM RIGID FRAMES
C 0000 0 JAMES FRIESEN OKLA STATE UNIV
     5 0 DIMENSION Y(8),SY(8),SYY(8),
     5 1 SXY(8),Q(4),G(8),Z(5)
    10 0 READ,C,C1,ADD,ADD1,TDP,TDE,K
    15 0 READ,N,N1,AX,TAX,BM,TBM,KIND
    16 0 READ,AA,AB,AC,AD,AE,AF
    17 0 NE=1
    20 0 P=N+N1
    25 0 SX=0
    30 0 SXX=0
    35 0 DO51J=1,8
    40 0 Y(J)=0
    45 0 SY(J)=0
    50 0 SYY(J)=0
    51 0 SXY(J)=0
    52 0 DO175I=1,N
    55 0 READ,U,Z(1),Z(2),Z(3),Z(4),
    55 1 Z(5),CODE
    56 0 BC=U-ADD
    60 0 X=BC*C
    65 0 SX=SX+X
    70 0 SXX=SXX+(X*X)
    75 0 GO TO(110,80),K
    80 0 TP=TDP*X
    81 0 TE=TDE*X
    82 0 W1=AB-Z(1)
    83 0 W2=AC-Z(2)
    84 0 W3=AD-Z(3)
    85 0 W4=Z(4)-AE
    86 0 W5=Z(5)-AF
    90 0 Y(1)=W1-(W2+W3)/2.0
    95 0 Y(2)=(W4+W5)/2.
   100 0 PUNCH,CODE,Y(1),TP,Y(2),TE,X
   105 0 GO TO 155
   110 0 TEBM=TBM*X
   115 0 TEAX=TAX*X
   120 0 DO126L=5,7,2
   125 0 G(L)=.0010-(Z(L-4)+Z(L-3))/2.0
   126 0 Y(L)=G(L)*AX
   127 0 IF(Y(7))128,130,128
   128 0 CA=2.
   129 0 GO TO 131
   130 0 CA=1.
   131 0 Y(3)=(Y(5)+Y(7))/CA
   135 0 DO141L=6,8,2
   140 0 G(L)=Z(L-5)-(Z(L-5)+Z(L-4))/2.
   141 0 Y(L)=G(L)*BM
   142 0 IF(Y(8))143,145,143
   143 0 CB=2.
   144 0 GO TO 146
   145 0 CB=1.
   146 0 Y(4)=(Y(6)+Y(8))/CB
   150 0 PUNCH,CODE,Y(3),TEAX,Y(4),
   150 1 TEBM,X
   155 0 CONTINUE
   160 0 DO175L=1,4,1
   165 0 SY(L)=SY(L)+Y(L)
   170 0 SYY(L)=SYY(L)+(Y(L)*Y(L))
   175 0 SXY(L)=SXY(L)+(Y(L)*X)
   180 0 IF(N1)205,205,181
   181 0 GO TO (182,205),NE
   182 0 NE=2
   185 0 C=C1
   190 0 N=N1
   191 0 ADD=ADD1
   200 0 GO TO 52
   205 0 Q(1)=TDP
   210 0 Q(2)=TDE
   215 0 Q(3)=TAX
   220 0 Q(4)=TBM
   234 0 DEMON=SXX-(SX*SX)/P
   236 0 IF(K-1)285,240,237
   237 0 KK=1
   238 0 KKK=2
   239 0 GO TO 243
   240 0 KK=3
   241 0 KKK=4
   242 0 GO TO 243
   243 0 DO275M=KK,KKK,1
   245 0 TOP=SXY(M)-(SY(M)*SX)/P
   250 0 B=TOP/DEMON
   255 0 A=(SY(M)/P)-B*(SX/P)
   260 0 BOT=SYY(M)-(SY(M)*SY(M))/P
   262 0 IF(BOT)265,263,265
   263 0 RR=99999999
   264 0 GO TO 270
   265 0 RR=(B)*(TOP/BOT)
   270 0 W=Q(M)
   275 0 PUNCH,A,B,W,RR,M,KIND
   280 0 GO TO 10
   285 0 END
```

TABLE A.II

FORTRAN PROGRAM USED TO CALCULATE STANDARD DEVIATION OF THE MEAN AND STUDENT'S T VALUES

```
C 0000 0 PROJECT 633
C 0000 0 OKLAHOMA STATE UNIVERSITY
C 0000 0 JAMES FRIESEN
     5 0 READ,N,KODE
    10 0 DF=N-1
    11 0 M=N-1
    12 0 SB=0.0
    13 0 SBB=0.0
    15 0 DO30L=1,N,1
    20 0 READ,A,B,W,RR,J,KIND
    25 0 SB=SB+B
    30 0 SBB=SBB+(B*B)
    31 0 P=N
    35 0 TOP=SBB-(SB*SB)/P
    40 0 SXX=TOP/DF
    45 0 SX=SQRTF(SXX)
    50 0 SXXB=SXX/P
    55 0 SXB=SQRTF(SXXB)
    60 0 XB=SB/P
    65 0 T1=(XB-W)/SX
    70 0 T2=(XB-W)/SXB
    71 0 NA=1
    72 0 NB=2
    75 0 PUNCH,XB,SX,T1,M,NA,KODE
         END
```

APPENDIX B

EXPERIMENTAL AND THEORETICAL

RESULTS

Explanation of Coding Scheme and Table Contents

Code: The coding scheme for Tables B.I, B.II, and B.III

Example:

11 11 11

- Position on frame at which measurements were made.
  - 01 = deflection at peak
  - 02 = deflection at haunch
  - 11, 22, 33, 44, and 55 = strain measurements and positions are shown in Figure 20.
- Type of frame loading
  - 11 = gravity loading
  - 22 = support rotation of hingeless frame
  - 33 = translational support movement of hingeless frame
  - 44 = lateral support movement of two-hinged frame
- Type of frame
  - 11 = hingeless frame
  - 22 = two-hinged frame
  - 32 = two-hinged frame with end fixed
  - 42 = two-hinged frame with extra fastenings between member sections

Mean Value:* The arithmetic mean of the regression values for each cycle of loading.

Theoretical Values:* The theoretical value determined analytically.

*Note. The above values are given as:
- Measurements
  - Deflection = in/unit of loading
  - Axial Load = lb/unit of loading
  - Bending Moment = ft-lb/unit of loading
- Loading
  - Gravity = lb total applied to frame

SUPPORT ROTATION = RADIANS AT EACH SUPPORT
TRANSLATIONAL MOVEMENTS = FT AT EACH SUPPORT
LATERAL MOVEMENT = FT AT EACH SUPPORT

PERCENT DIFFERENCE: THE PERCENT DIFFERENCE BETWEEN THE MEAN VALUE AND THEORETICAL VALUE BASED ON THE THEORETICAL VALUE.

STANDARD DEVIATION OF THE MEAN: THE STANDARD DEVIATION FOR THE MEAN VALUE OF THE REGRESSION VALUES FOR EACH CYCLE OF LOADING.

PROBABILITY: THE PROBABILITY OF THE EXPERIMENTAL VALUE NOT HAVING THE SAME VALUE AS THE THEORETICAL VALUE.

TABLE B.1

DEFLECTION RESULTS AT THE PEAK
AND HAUNCH

CODE	MEAN VALUE	THEORETICAL VALUES	PERCENT DIFFERENCE	STANDARD DEVIATION OF THE MEAN	PROB-ABILITY
111101	45.600×10^{-5}	34.170×10^{-5}	+33.45	1.3999×10^{-5}	100
113301	10.948	8.5500	+28.06	.16591	100
112201	42.987	59.300	-27.51	.78014	100
221101	59.929×10^{-5}	52.640×10^{-5}	+13.85	$.66678 \times 10^{-5}$	100
321101	57.810×10^{-5}	52.640×10^{-5}	+ 9.82	$.32414 \times 10^{-5}$	100
224401	6.6833	9.0400	-26.07	.19790	100
421101	52.995×10^{-5}	52.640×10^{-5}	+ .67	$.28332 \times 10^{-5}$	<60
111102	13.721×10^{-5}	11.300×10^{-5}	+21.42	$.35312 \times 10^{-5}$	100
113302	3.4066	4.1810	-18.52	.12301	100
112202	13.3217	19.640	-32.18	.35550	100
221102	17.781×10^{-5}	17.460×10^{-5}	+ 1.84	$.16499 \times 10^{-5}$	95.9
321102	16.954×10^{-5}	17.460×10^{-5}	- 2.90	$.13800 \times 10^{-5}$	99.3
224402	2.4046	2.3920	+ 0.53	.36381	<60
421102	15.771×10^{-5}	17.460×10^{-5}	- 9.67	$.24296 \times 10^{-5}$	90.5

TABLE B.11

RESULTS OF AXIAL FORCES IN FRAME MEMBERS

CODE	MEAN VALUE	THEORETICAL VALUES	PERCENT DIFFERENCE	STANDARD DEVIATION OF THE MEAN	PROB-ABILITY
111111	.71433	.50000	+42.87	.25903	78.5
111122	.53304	.50000	+ 6.61	.16683	< 60
111133	.56571	.70890	-20.20	.19227	76.1
111144	.18362	.70890	-74.19	.17635	99.3
111155	.35220	.50000	-29.56	.08860	92.9
112211	-9359.6	0		19340	67.6
112222	30311	0		14666	95.6
112233	-30607	-52324	-41.51	16795	87.2
112244	-40881	-52324	-21.87	11930	81.0
112255	23834	0		13278	92.2
113311	-1468.4	0		1481.3	81.6
113322	1750.8	0		475.55	99.5
113333	-4604.2	-5611.3	-17.95	484.22	95.7
113344	-8977.8	-5611.3	+59.99	825.77	99.7
113355	467.77	0		782.75	70.8
221111	.52219	.50000	+ 4.44	.03724	71.5
221122	.60451	.50000	+25.34	1.1222	81.1
221133	.48265	-.27695	+74.27	.05591	99.7
221144	-.08833	-.27695	-68.11	.03946	100
224411	-693.59	0		1033.8	83.1
224422	584.29	0		376.88	91.4
224433	41.911	-7.9801	+425.19	1285.0	<60
224444	-4458.6	-7.9801	+55771	1618.1	98.4
321111	.53890	.50000	+ 7.78	.02461	91.0
321122	.50943	.50000	+ 1.88	.06638	<60
321133	.51681	.27695	+86.61	.03423	100
321144	-.03681	-.27695	-86.71	.11431	97.8

TABLE B.III

RESULTS OF BENDING MOMENTS
IN FRAME MEMBERS

CODE	MEAN VALUE	THEORETICAL VALUES	PERCENT DIFFERENCE	STANDARD DEVIATION OF THE MEAN	PROB-ABILITY
111111	1.0958	1.5158	-27.71	.03873	100
111122	-1.4705	-1.3949	+ 5.42	.02098	99.6
111133	-1.6692	-1.4520	+14.96	.02584	100
111144	1.0874	.82610	+31.63	.03164	100
111155	.38882	.55598	-30.07	.03062	100
112211	-407112	-523520	-22.24	5093.4	100
112222	-117805	-153500	-23.25	2474.0	100
112233	- 55354	- 55681	- 0.59	1974.6	<60
112244	118884	149887	-20.68	2769.2	100
112255	-305533	-401760	-23.95	3116.5	100
113311	- 38843	- 46862	-17.11	1255.0	100
113322	-4792.3	-7138.0	-32.86	599.94	99.6
113333	4776.1	3352.2	+42.48	668.17	95.6
113344	23596	25398	-11.70	372.32	99.8
113355	- 28168	- 33763	-16.57	1165.0	99.5
221111	-.23303	-.23535	- 0.99	.00442	100
221122	-1.8165	-1.8734	- 3.04	.00633	100
221133	-1.8812	-1.6256	+15.77	.02112	100
221144	1.3944	1.2935	+ 7.80	.01483	100
224411	1223.6	840.93	+45.51	133.69	98.1
224422	7261.6	7205.8	+ 0.77	243.13	<60
224433	10127	8350.4	+21.28	242.41	100
224444	12152	11832	+ 2.71	586.01	70.0
321111	.15257	.23535	-35.18	.00251	100
321122	-1.8650	-1.8734	- 0.45	.01051	76.8
321133	-1.9023	-1.6256	+17.02	.00905	100
321144	1.3937	1.2935	+ 7.75	.00959	100

APPENDIX C

ANALYSIS OF VARIANCE TABLES

TABLE C.I

ANALYSIS OF VARIANCE IN PRELIMINARY TESTS DUE TO DEFLECTION

Source	Degrees Freedom	Sum of Squares	Mean Square	Variance Ratio	Sign. Level
TOTAL	15	.00101194			
FRAME MEMBERS	3	.00096158	.00032053	59.756	100
CYCLE	1	.00000812	.00000812	1.5138	73.0
LOADING	1	.00000484	.00000484	.9023	
ERROR	10	.00005364	.000005364		

TABLE C.II

ANALYSIS OF VARIANCE IN PRELIMINARY TESTS DUE TO STRAIN GAGE READINGS

Source	Degrees Freedom	Sum of Squares	Mean Square	Variance Ratio	Sign. Level
TOTAL	31	.00537082			
FRAME MEMBERS	3	.00040541	.00013514	.7494	
GAGE POSITION	1	.00041760	.00041760	2.3156	83.2
CYCLE	1	.00003916	.00003916	.2171	
LOADING	1	.00000015	.00000015	.00083	
ERROR	25	.0045085	.00018034		

TABLE C.III

ANALYSIS OF VARIANCE DUE TO DEFLECTION AT THE PEAK

Source	Degrees Freedom	Sum of Squares	Mean Square	Variance Ratio	Sign. Level
Combination of Hingeless and Two-Hinged					
Treatment total	23	13.9765×10^{-8}			
End condition	1	12.3229×10^{-8}	12.3229×10^{-8}	14.314	99.5
Load cycle	2	$.1243 \times 10^{-8}$	$.06215 \times 10^{-8}$	.07219	
Interaction	2	1.5293×10^{-8}	$.76465 \times 10^{-8}$	.8882	55.9
Error	18	15.4965×10^{-8}	$.8609 \times 10^{-8}$		
Hingeless					
Total	11	2.5860×10^{-8}			
Frame	3	$.7488 \times 10^{-8}$	.23960	4.03	92.5
Cycle	2	1.4801×10^{-8}	.74005	12.44	99.7
Error	6	$.3571 \times 10^{-8}$	.05951		
Two-Hinged					
Total	11	58.693			
Frame	3	20.506	6.835	1.97	75.0
Cycle	2	17.360	8.680	2.50	80.3
Error	6	20.827	3.471		

TABLE C.IV

ANALYSIS OF VARIANCE DUE TO
DEFLECTION AT THE HAUNCH

Source	Degrees freedom	Sum of squares	Mean square	Variance ratio	Sign. Level
Combination of Hingeless and Two-Hinged					
treatment total	23	1.12947×10^{-8}			
end condition	1	$.98945 \times 10^{-8}$	$.98945 \times 10^{-8}$	293.7797	100
load cycle	2	$.09179 \times 10^{-8}$	$.045895 \times 10^{-8}$	13.627	100
interaction	2	$.04823 \times 10^{-8}$	$.024115 \times 10^{-8}$	7.16	99.5
error	18	$.06063 \times 10^{-8}$	$.003368 \times 10^{-8}$		
Hingeless					
total	11	258.684			
frame	3	74.888	24.963	4.18	93.0
cycle	2	148.006	74.003	12.41	99.7
error	6	35.790	5.965		
Two-Hinged					
total	11	3.594			
frame	3	2.252	.7504	8.67	98.5
cycle	2	.823	.4115	4.76	93.9
error	6	.519	.0865		

TABLE C.V

ANALYSIS OF VARIANCE DUE TO BENDING MOMENTS AT POSITION 22

Source	Degrees Freedom	Sum of Squares	Mean Square	Variance Ratio	Sign. Level
Combination of Hingeless and Two-Hinged					
Treatment total	23	.78186			
End condition	1	.71843	.71843	362.84	100
Load cycle	2	.02011	.01005	5.076	98.3
Interaction	2	.00767	.00384	1.939	80.5
Error	18	.03565	.00198		
Hingeless					
Total	11	.05813			
Frame	3	.01256	.00417	1.30	62.7
Cycle	2	.02630	.01315	4.10	91.9
Error	6	.01927	.00321		
Two-Hinged					
Total	11	.00530			
Frame	3	.00206	.00103	3.55	90.9
Cycle	2	.00148	.00074	2.55	80.8
Error	6	.00176	.00029		

TABLE C.VI

ANALYSIS OF VARIANCE DUE TO BENDING MOMENTS AT POSITION 33

Source	Degrees Freedom	Sum of Squares	Mean Square	Variance Ratio	Sign. Level
Combination of Hingeless and Two-Hinged					
treatment total	23	.41670			
end condition	1	.26969	.26969	44.43	100
load cycle	2	.02781	.01390	2.29	85.0
interaction	2	.09860	.04930	8.13	99.6
error	18	.10934	.00607		
Hingeless					
total	11	.08812			
frame	3	.02716	.00905	1.87	74.9
cycle	2	.03191	.01596	3.30	88.4
error	6	.02905	.00484		
Two-Hinged					
total	11	.05889			
frame	3	.02524	.00841	1.81	73.2
cycle	2	.00575	.00288	.62	
error	6	.02790	.00465		

TABLE C.VII

ANALYSIS OF VARIANCE DUE TO BENDING MOMENTS AT POSITION 44

Source	Degrees freedom	Sum of squares	Mean square	Variance ratio	Sign. level
Combination of Hingeless and Two-Hinged					
treatment total	23	.7267			
end condition	1	.5655	.5655	65.60	100
load cycle	2	.00019	.000095		
interaction	2	.00580	.00290	.34	
error	18	.15521	.00862		
Hingeless					
total	11	.13218			
frame	3	.10330	.03443	7.79	98.1
cycle	2	.00236	.00118	.27	
error	6	.02652	.00442		
Two-Hinged					
total	11	.02904			
frame	3	.02194	.00731	12.60	99.5
cycle	2	.00361	.00186	3.21	87.5
error	6	.00349	.00058		

TABLE C.VIII

ANALYSIS OF VARIANCE DUE TO PINNED AND RESTRAINED SUPPORTS OF TWO-HINGED FRAMES

Source	Degrees Freedom	Sum of Squares	Mean Square	Variance Ratio	Sign. Level
Deflection at the Peak					
Pinned vs. restrained	1	17.978×10^{-5}	17.978×10^{-5}	4.65	95.2
Pinned + restrained	16	61.849×10^{-5}	3.866×10^{-5}		
Deflection at the Haunch					
Pinned vs. restrained	1	2.938×10^{-5}	2.938×10^{-5}	9.73	99.3
Pinned + restrained	16	4.825×10^{-5}	$.302 \times 10^{-5}$		

VITA

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