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A study of the composite action of light gage steel deck and concrete

Charles Sherman Bach Jr.

A STUDY OF THE COMPOSITE ACTION OF LIGHT
GAGE STEEL DECK AND CONCRETE

BY

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THESIS

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Abstract

This thesis is on the investigation of corrugated light-gage steel sheets used as the tension reinforcement for concrete slabs. The steel sheets used had embossments placed at uniform intervals over the entire length of the sheets. The composite concrete and light-gage deck steel were tested to determine a method of design for this type of composite section from an economical, safe and practical standpoint. The investigation covered the flexural, shear and bond capacity of this type of composite section.

ACKNOWLEDGEMENTS

The invaluable aid given by J. W. Hubler, Professor of Civil Engineering; Dr. Wei-Wen Yu, Associate Professor of Civil Engineering; Martin M. Mitchum, Laboratory Manager and Robert McKee, Graduate Research Assistant made the completion of this thesis possible. The financial aid and steel deck from Wheeling Corrugating Company were appreciated. My wife, Judith M. Bach was the inspiration which helped complete this thesis.

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I. INTRODUCTION

In the past few years, the uses of light gage steel have increased. Code and specification writers have been hard pressed to keep up with the new developments in this field. One of the developments has been the use of the composite action of steel deck and concrete which is not entirely a new subject. The idea of placing embossments in the steel to provide a better shear and bond relation between the steel and concrete is new. The problem is to determine the most economical and easy method of design for this type of composite action. Many methods have been suggested which follow the various codes such as the A.C.I., A.I.S.C. and the A.I.S.I. codes. The decision on the method of design depends on the test results.

The reason why this type of composite action is being considered is the amount of time and money it will save on an actual job. With this type of construction, the steel deck can serve as the form for the concrete and also serve as the tension reinforcement. In this manner, it will save time because there is no need to tear down the forms after the concrete is set. There will also be a saving of time in setting up the steel deck in order to place the concrete since the only work that needs to be done is placing shoring under the spans and providing reinforcement over the supports. Another reason for investigating this type of composite action is concrete customarily is used as a filler for steel deck (acts as an insulator), therefore it is natural to have these materials act together to provide additional dependable strength.

Research of this type is needed so that engineers will be able to design better buildings at lower cost than previously was possible. Through this research, they may obtain design methods which are easy enough for the average engineer to use, but which are accurate enough so that this type of design is not too conservative to be practical.

The type of deck used was Wheeling Corrugating Company's Type B deck (see Fig. 1) with embossments. Although only one type of steel deck was used, the results of this thesis should be applicable to similar types of steel deck. Therefore, this research will serve more people than just the customers of Wheeling Corrugating Company.

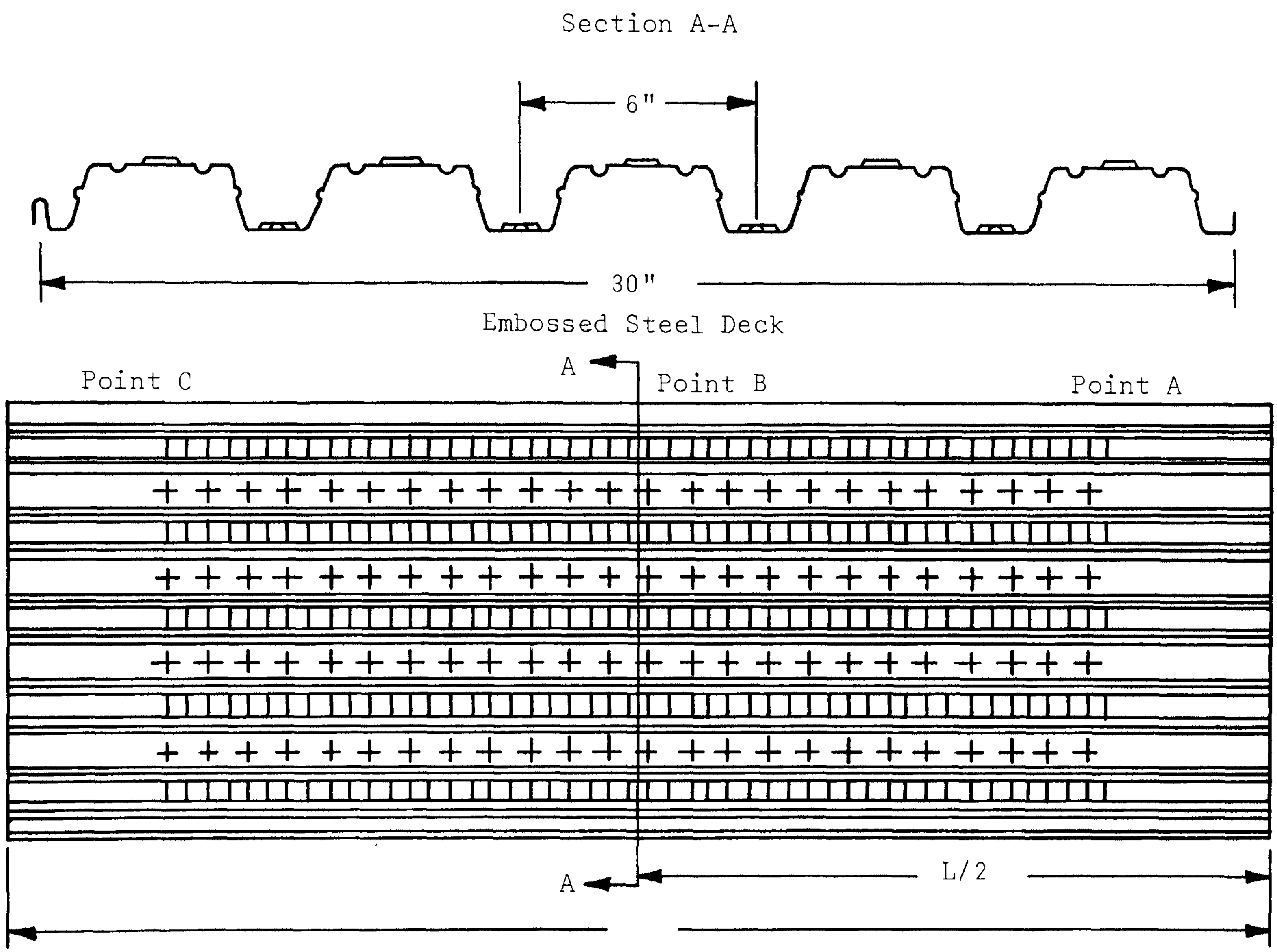


Fig. 1 Wheeling Corrugating Type B Deck

II. LITERATURE REVIEW

There is no published literature applying directly to the subject investigated. There is considerable literature on ultimate strength design in concrete and on composite design of steel beams and concrete. There also is a series of articles on the capacity of the connections between light gage steel deck and concrete composite over an end beam. The author tested the strength increase of the end beam to see what the results were.(1) This does not apply directly to our flexural problem, but it does help in determining how to provide end anchorages.

The literature on the ultimate strength design is quite extensive. It seems that ultimate strength design was first proposed in the late eighteen hundreds, but at the time working stress design took precedence because it was thought to be the best method. In the nineteen thirties and forties extensive research was done on the problem of ultimate strength design in many parts of the world. Many researchers suggested various types of concrete stress distribution as shown in Fig. 2. The distribution finally used as the correct one was the distribution that matched the stress-strain diagram of concrete. Whitney and others then approximated this by a rectangular stress distribution.(2)

There have been attempts to improve on this rectangular distribution. All such attempts refer to the stress distribution of the stress-strain relation.

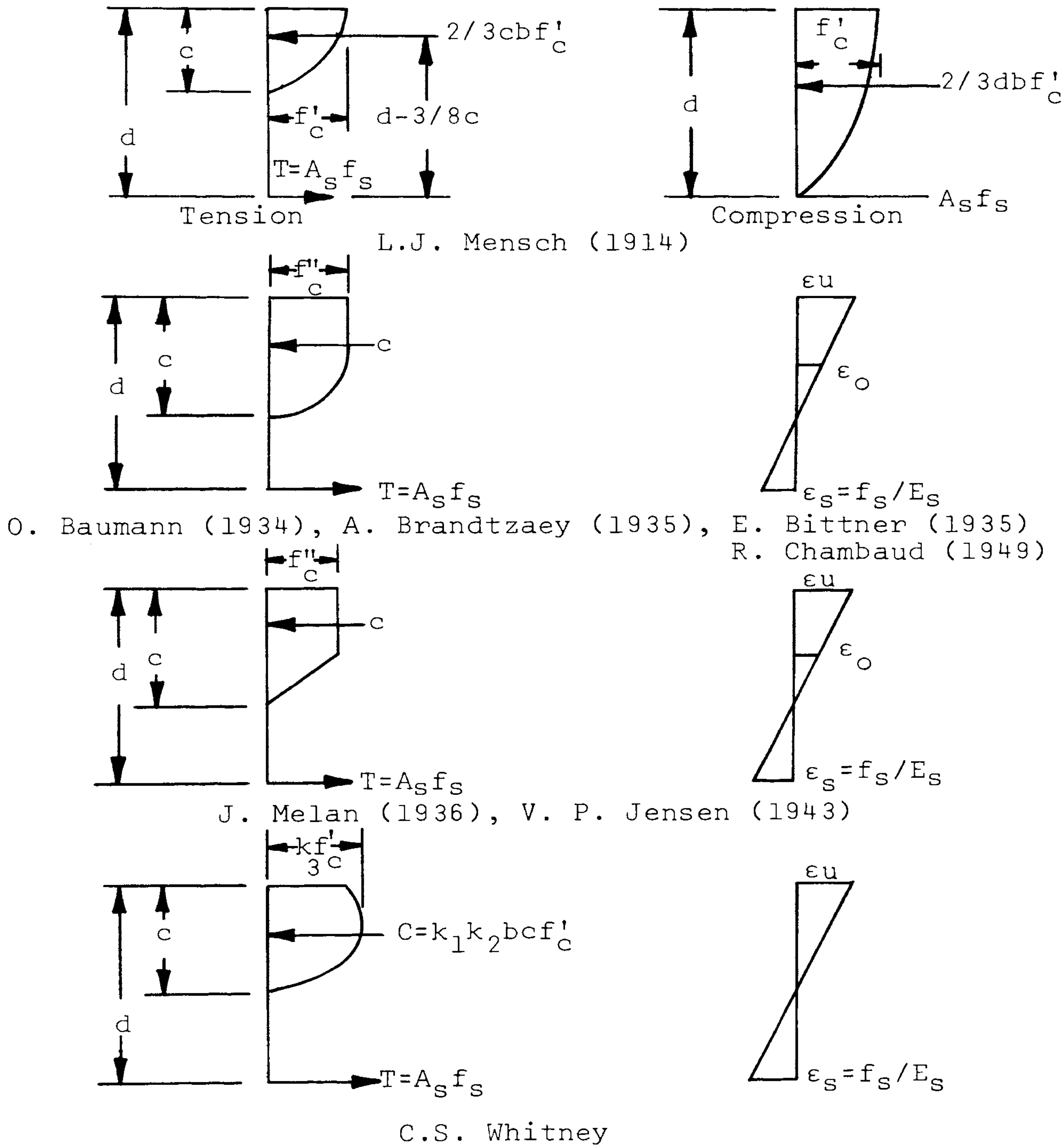


Fig. 2 Ultimate Strength Stress Distributions

There is a considerable volume of literature on the composite action of steel beams and concrete. One commercial producer advocates this method as the one to use; but tests results will be checked to determine whether this method applies to our situation. This principle can be applied to sections of shallow concrete depth and relatively large overall depths of light gage steel.

The literature on working stress design is very old and this method is covered quite accurately in the current A.C.I. code.

There is a private report done by the Iowa State University Testing Station for the A.I.S.I. committee on light gage steel but they were testing more on the bond strength than on the flexural strength of light gage steel composite. This report has been examined and found to be conservative in its design procedures.

Several companies are marketing this type of product with various types of design procedures for each company's product. Since each product has a different arrangement of embossments and shapes, the load characteristics are different for each type of corrugated steel. The design procedures given in this paper apply to the Wheeling Corrugating's series B steel deck with its unique arrangement of embossments.

III. EXPERIMENTAL PROCEDURE

In order to have accurate results in a testing program, there must be a definite procedure: before, during and after the tests have been completed. In our program, the first step was to obtain all the important measurements on the steel deck which the Wheeling Corrugating Company had shipped. Measurements were taken of the height of the embossments in the embossed steel deck. To obtain embossment heights, a linear, variable, differential transformer calibrated to read inches was used. This device was used to measure the height of the embossments which were on the horizontal surfaces of the deck. The embossments on the vertical surfaces were measured using an inside caliper. On each piece of embossed deck, the embossments at A, B, and C, as shown on Fig. 1, were measured. On every piece of plain and embossed deck, the width, metal thickness, depth of corrugations and total length were measured. Metal thickness was measured with a micrometer. The remainder of the measurements were completed with a tape measure which was accurate to the nearest sixteenth of an inch. The width and depth of corrugations were measured at each end of the deck. The thickness was measured at points A, B and C as shown in Fig. 1.

Once all the measurements were obtained, the decks were numbered with an electronic engraver. An example of this data is shown in Table I.

Specimen Data for Deck No. 10

Nominal Gage: 20 (0.0396 in.)
Overall Length: 10'-11 7/8"
Test Span: 10'-0"

Point	A	B	C
Width-in.	30 1/8		30
Thickness in.	0.0390	0.0395	0.039
E-1 in.	0.101	0.094	0.102
Depth-in.	1 1/2		1 7/16
E-2 in.	0.063	0.075	0.071
E-3 in.	0.08	0.08	0.09

Table I Specimen Data

The physical measurements on the deck were used to set up a concrete placing schedule. The schedule included each piece of deck, the depth of concrete and the approximate time of placing. In Table II, the placing schedule is illustrated for the ten foot test spans.

Before the concrete was placed, care was taken to obtain the correct depth of concrete for each particular specimen. While the concrete was being placed, the slump of the concrete was taken and representative test cylinders were cast to determine the strength of the concrete.

The concrete was placed in forms made of two by twelve boards braced at the top and bottom. The proper depth of concrete was obtained by building up the supports under the deck until the concrete could be leveled off at the top of the forms. The supports were placed at points where a minimum of stresses were introduced into the steel deck due to the wet concrete dead load. For example, the supports were placed at the one third points for a test span of ten feet. An example of the forms can be seen in Fig. 3.

After the concrete was placed, leveled and troweled, we allowed it to obtain its initial set. After the concrete set, it was covered with plastic to retain the moisture. Everyday for twenty-eight days, the concrete was wetted. It was covered for the entire twenty-eight days. This procedure was followed in an attempt to cure the concrete at its optimum curing condition which is one hundred

Test Schedule

Test No.	Span (ft.)	Overall Thickness	Gage	Type of Test
1	10	4½	20	Regular-Ungreased
2	10	4½	20	Regular-Ungreased
3	10	4½	20	Embossed-Greased
4	10	4½	20	Embossed-Greased
5	10	4½	20	Embossed-Ungreased
6	10	4½	20	Embossed-Ungreased
7	10	3½	20	Regular-Ungreased
8	10	3½	20	Embossed-Greased
9	10	3½	20	Embossed-Ungreased
10	10	3½	20	Embossed-Ungreased
11	10	5½	20	Embossed-Greased
12	10	5½	20	Regular-Ungreased
13	10	5½	20	Embossed-Ungreased
14	10	5½	20	Embossed-Ungreased
15	10	5	20	Regular-Ungreased
16	10	5	20	Embossed-Ungreased
17	10	4½	22	Embossed-Ungreased
18	10	4½	22	Embossed-Ungreased
19	10	4½	22	Regular-Ungreased
20	10	4½	22	Embossed-Greased
21	10	4½	16	Embossed-Ungreased
22	10	4½	16	Embossed-Ungreased
23	10	4½	16	Regular-Ungreased
24	10	4½	16	Embossed-Greased
25	10	4½	18	Embossed-Ungreased

Table II Test Schedule

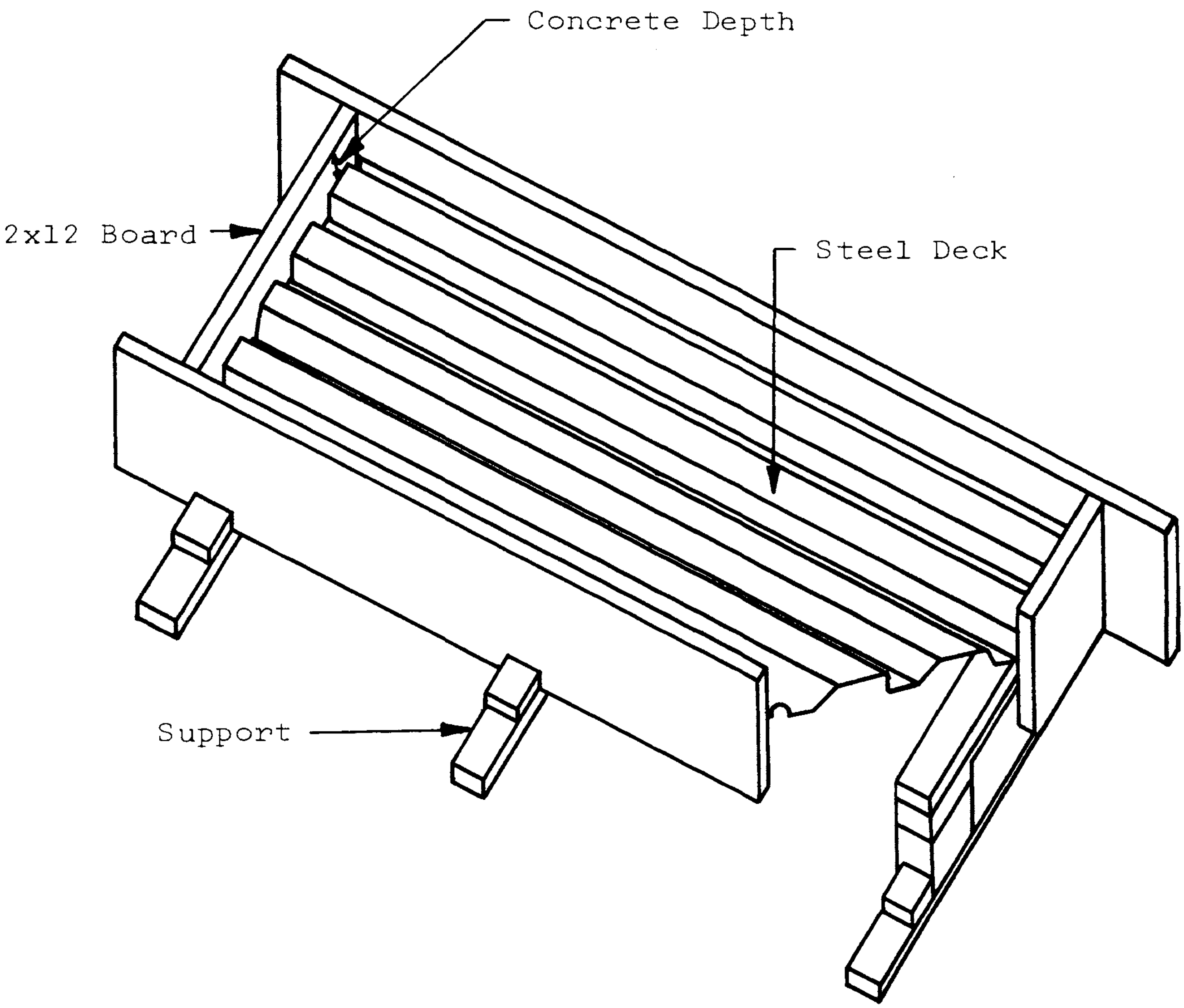
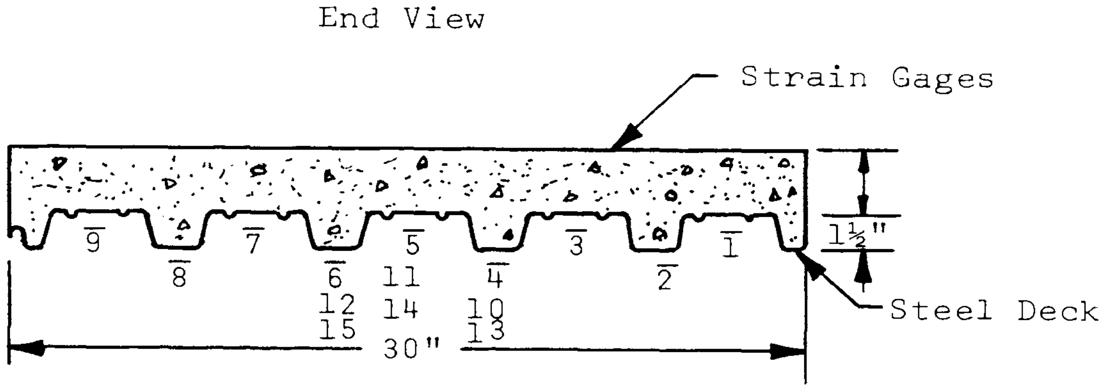


Fig. 3 Concrete Forms

percent humidity and seventy to seventy-five degrees fahrenheit. The forms were removed after one week and the composite beams moved to allow for the next pour.

After the twenty-eight day period, the composite beams were ready to be tested. During this time, an arrangement of strain gages was determined. The strain gage pattern included three SR-4 A-9 strain gages on the concrete (top of the slab) at the center of the test span. These gages were placed seven and one half inches from the edge and at the center of the concrete. The strain gages on the steel (bottom of the slab) were placed at the center and at the one sixth points of the test span. At the center, nine strain gages were placed across the steel deck as shown in Fig. 4. Three strain gages were placed at the one sixth point from each support. The location of these gages can be seen in Fig. 4. The numbering system is shown in Fig. 4.

The strain gages were very carefully placed to insure accurate readings. All of the strain gages were connected to the automatic strain gage equipment. This equipment consisted of one Datran II strain gage balancing unit, one Datran II switching unit and one Franklin printer. This equipment was capable of reading one gage every one tenth of a second to one gage every second. The fastest speed was the least accurate, therefore the slowest speed was used for these tests. This speed was much faster than that obtained by switching and balancing manually.



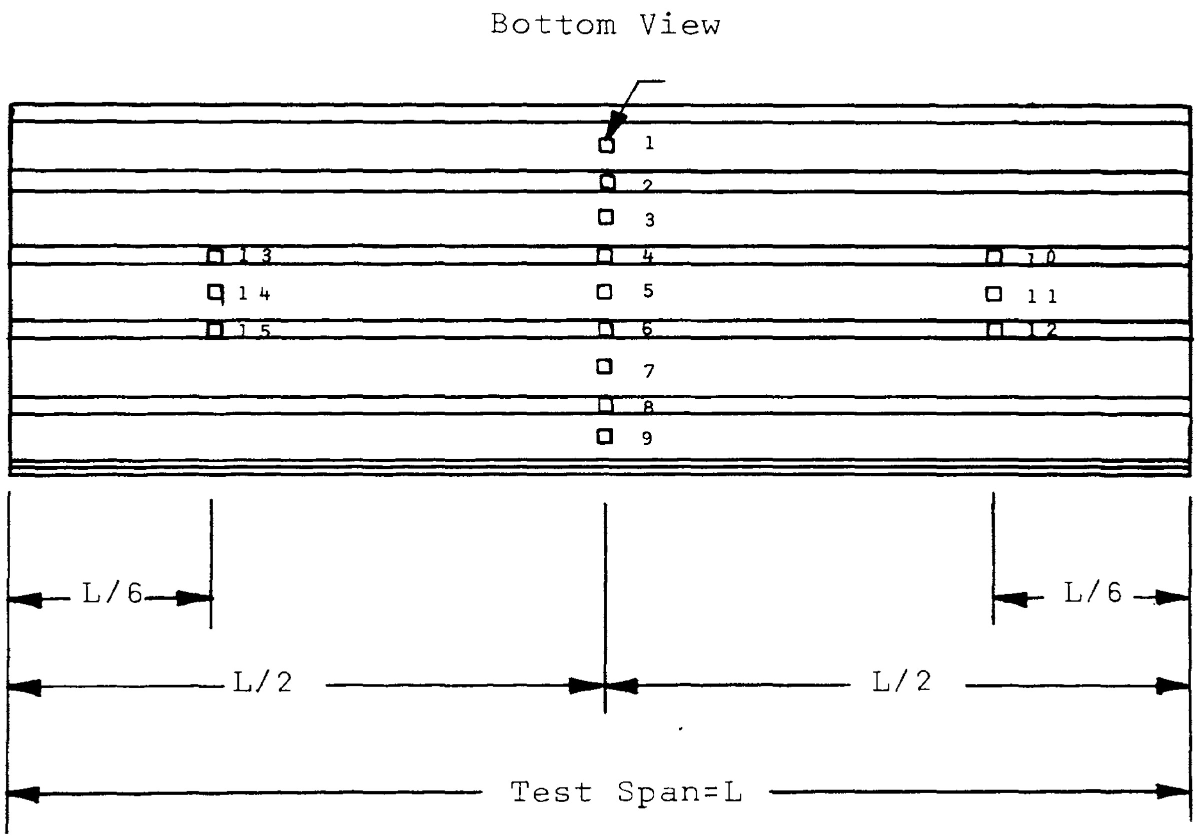


Fig. 4 Strain Gage Location

The composite beam was prepared for a test by placing one inch rollers under the ends of the beam. Loads were applied at the one third points of the test span. At these points, a three inch steel plate was embedded in hydra-stone to obtain a level loading on the concrete because it was difficult to obtain a perfectly flat concrete surface for loading. The load was applied to the three inch plate by a one inch roller which was welded to a seven inch I-beam on which the hydraulic jack applied the load. The load was measured by a calibrated load cell placed between the jack and the load frame. The load cell was read on the automatic strain gage equipment. The loading arrangement is shown in Fig. 5.

The deflections were measured by dial gages which had a three inch gage length. The deflections were measured at the center of test span. Two dial gages were used, one at each side of the composite beam, to detect any twist in the specimen.

Before a test began, a load increment was determined to obtain the most accurate results. After the first few tests, the load increment was increased so that the time for one test could be shortened. The load increments were applied at five minute intervals to allow for distribution of stress in the beam. This followed an A.S.T.M. E-6 standard. The strains and deflections were read at the end of each five minute period. These deflections and strain readings were used to obtain the results and conclusions.

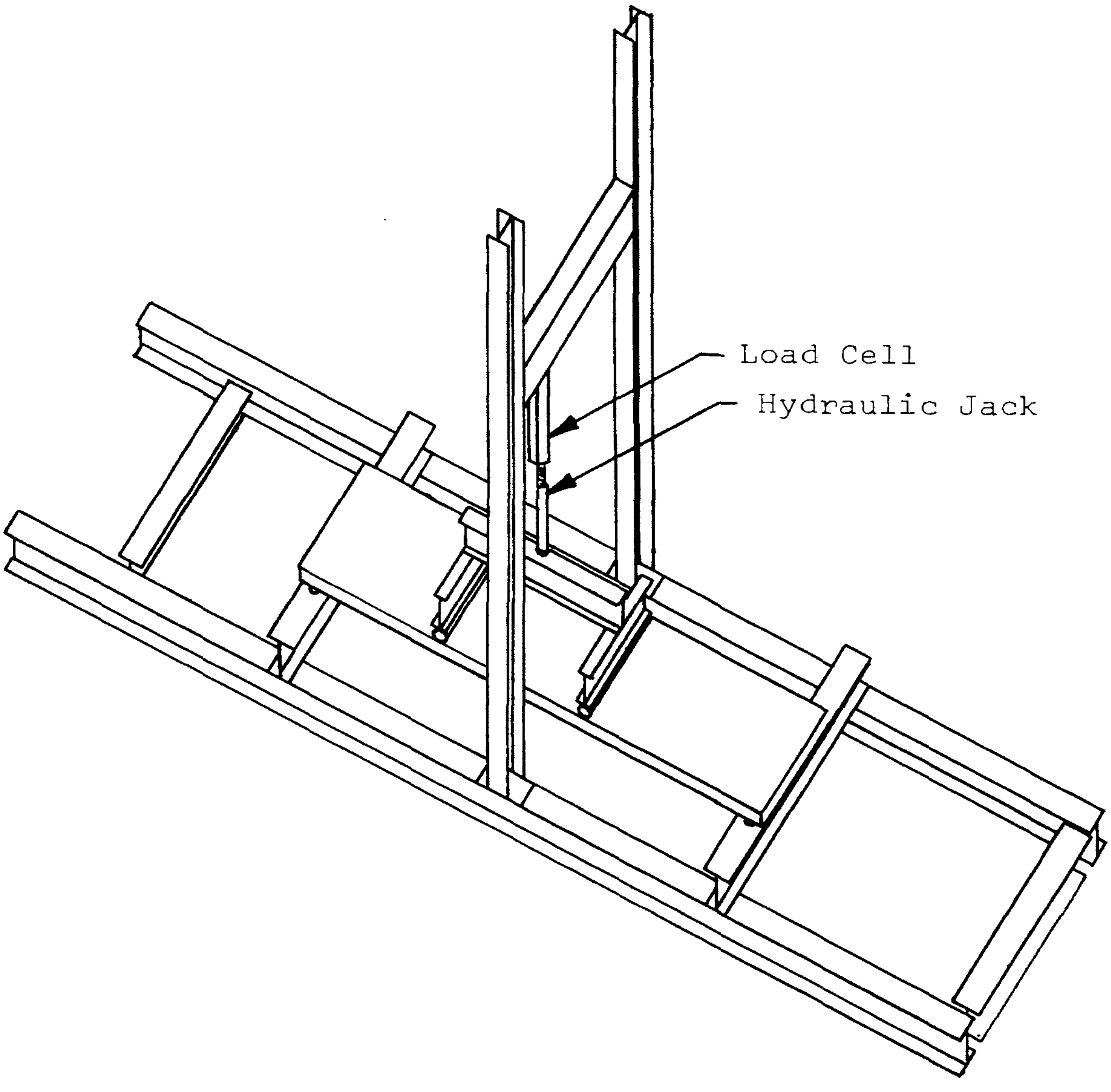


Fig. 5 Load Frame

IV. EXPERIMENTAL RESULTS

The results shown in this thesis were taken from a number of tests but only the test specimens shown in Table III were used as examples. The test specimens used in this paper were representative of the entire testing program.

The test data taken on all beams consisted of strains and deflections at specific loadings. The strain data was converted to stress by multiplying the strains by the modulus of elasticity of the material (steel or concrete) where the strain was measured. The stress data was then plotted against the loads. Deflections were plotted directly against the loads. The load deflection curve for one representative test specimen is shown in Appendix A (the solid line is the test curve). Stresses are plotted versus loads. The stress data for the specimens is plotted for each strain gage. Then the stress from the strain gages which are in similar positions are averaged into one curve. This gives four separate load versus stress curves for each specimen. An example of these types of curves is shown in Appendix I.

Specimens were loaded at the one third points of the test spans in order to approximate the effects of a uniform load.

To analyze the test results, one needs to determine the ultimate test moment and the ultimate load for each specimen. This information is listed in Table III. The ultimate test load was taken from the load cell data. The

Specimen Number	f'_c psi	Concrete Weight Lb/Ft	$E_c x 10^6$ psi	n	Test Span-Ft.	Ultimate Test Load	Ultimate(Ft-Lb) Test Moment
69	6000	110	3.95	9.83	10	7835 Lb.	13,058
66	8400	110	3.41	8.31	6	5425 Lb.	5,425
9	4000	140	3.52	8.0	10	3900 Lb.	6,500
10	4000	140	3.52	8.0	10	4950 Lb.	8,250
1	4300	140	3.54	8.09	10	3200 Lb.	5,333
6	4300	140	3.54	8.09	10	4800 Lb.	8,000
68	6000	110	2.85	9.83	10	11,625 Lb.	19,375

Table III Test Data

ultimate moment was calculated by multiplying the test load divided by two and multiplied by one third the test span plus the dead load and test rig moments.

$$M_{ut} = \frac{P_{ut}}{2} X \frac{L}{3} X 12 + M_{test} + \frac{W_d L^2}{8} X 12$$

M_{test} = moment due to the test set up-inch pounds

M_{ut} = ultimate test moment-inch pounds

P_{ut} = ultimate test load-pounds

L = test span-feet

W_d = dead load of the test specimen

The modulus of elasticity for the steel was the American Concrete Institute recommended value of twenty-nine million (29,000,000 psi) pounds per square inch. The concrete modulus of elasticity was the American Concrete Institute's recommended value calculated by the formula:(3)

$$E_c = w^{1.5} X 33 X (f'_c)^{\frac{1}{2}}$$

E_c = concrete modulus of elasticity-pounds per square inch

w = weight of concrete-pounds per cubic feet

f'_c = ultimate concrete stress-pounds per square inch

Table III lists the modulus of elasticity and the ratio of steel modulus of elasticity to the concrete modulus of elasticity, n.

$$n = \frac{E_s}{E_c}$$

E_s = steel modulus of elasticity

V. ANALYSIS OF RESULTS

A method was formulated to predict the experimental results presented in the previous section. It was necessary for this method to have a sufficient factor of safety but still not be conservative and therefore waste material. Any number of methods were available to use such as the American Concrete Institute Codes working stress design and ultimate strength design.(3) Many such methods were tried but the one which gave the best results was a combination of working stress design and flexural stress theory.(4)

In the working stress design, the first operation is to determine the effective depth of concrete as compared to the steel area. This is determined by the formula:

$$k = \sqrt{(pn)^2 + 2pn} - pn$$

$p = A_s/bd$ = steel ration

A_s = area of steel

b = width of the composite beam

d = the distance from the top of concrete to the centroid of the steel

n = ratio of steel modulus of elasticity to the concrete modulus of elasticity

k = the fraction of the concrete depth to the d distance

kd = effective depth of concrete

The neutral axis of the beam is at the bottom of the effective concrete depth. Knowing the location of the neutral axis, the moment of inertia of the entire beam section can be determined. The formula for this operation is:

$$I_t = (kd)^3 (b)/3 + I_s(n) + A_s(n) (1-k)d^2$$

I_t = total beam moment of inertia

I_s = moment of inertia of the steel deck.

Knowing the total moment of inertia and the neutral axis of the beam, the deflection and resisting moment can be calculated for a composite beam. The stress theory is used to calculate the resisting moment and the working stress method used to check deflections, bond and shear. The formula used for the calculation of the resisting moment is:(4)

$$M_r = \frac{(f_s)(I_t)}{(n)(c)(12)}$$

f_s = allowable steel stress, which was 20,000 psi for this case

M_r = resisting moment

c = distance from the neutral axis to the outer edge of the material (in this case it was the bottom of the steel).

In Appendix A are sample calculations for specimen number 68 for resisting moment. Also in Appendix A, there is a calculation of stress at a load of three thousand pounds which is compared to the curves of the stress taken from the test data. These results show that the calculations give conservative results. A comparison of the ultimate test moment and the calculated moment is in Table IV. This table shows the adequate factor of safety for this method of calculation.

Table of Moments

Specimen Number	Ultimate Test Moment lb-ft/30 in.	Test Dead-Load Moment lb-ft/30 in.	Max. Test Moment lb-ft/12 in.	K	I_t-in.^{4}	Design Resisting Moment $\frac{\text{ft-lb}}{\text{12 in.}}$	Max. M. ÷ Dsn. M.
69	13,058	1381	5775	0.5427	29.976	2403	2.403
66(1)	5,424	616	2416	0.4316	19.148	1597	1.513
9	6,500	1621	3248	0.4254	18.639	1604	2.025
10	8,250	1621	3948	0.4254	18.639	1604	2.461
1(2)	5,333	1987	2928	0.3782	37.390	2433	1.203
6	8,000	1987	3995	0.3782	37.390	2433	1.642
68	19,375	1953	8531	0.4486	104.833	5132	1.662

(1) Shear Span
(2) Unembossed Deck

Table IV Moments

Deflection calculations are the same for any simple beam. Any method may be used which calculates deflection accurately such as moment-area or virtual work. The usual limiting value for deflection is L/360.[(5)] A sample deflection calculation for specimen number 68 is given in Appendix A.

Shear calculation for the beams are the same as working stress calculations. That is:[(7)]

$$v_c = V/bd$$

$$V = \text{shear at } d \text{ from the support.}$$

This v_c shall not exceed $\sqrt{1.1}\ f'_c$.

Bond is computed by the working stress design formula:[(5)]

$$u = V/\Sigma ojd$$

u = bond stress

V = shear

j = $1-k/3$

Σo = the amount of steel in contact with concrete

k = is previously defined.

The bond should not be larger than 500 psi[4].

Shear and bond calculations are shown in Appendix A. These tests indicate that these methods are feasible.

VI. CONCLUSIONS

The use of the working stress analysis to determine the centroid and moment of inertia of the steel and concrete appears to be a good procedure for composite steel deck and concrete. The method of calculating moments takes into account the highly stressed portions of the steel deck at the farthest point from the neutral axis. The factor of safety for this method, about 1.65, is comparable to those usually used.

The deflections, calculated and observed, are in close relation to each other within the working stress range.

Steel controls the design in most cases due to the distance from the bottom of the deck to the neutral axis and the relatively small percentage of steel. Failures were generally gradual because of the ductility of the steel.

The theory presented here was conservative because the ends were unrestrained and in an actual installation the ends are welded. The ends are thus restrained by the adjacent beams. In an actual case the loads are concentrated and distributed laterally.

Additional tests need to be conducted on this type of composite section to determine if less conservative methods may be devised to design the beams. It is recommended to use shorter test spans to determine shear capacity and bond strength.

VII. APPENDICES

APPENDIX A

Calculations for Specimen No. 69

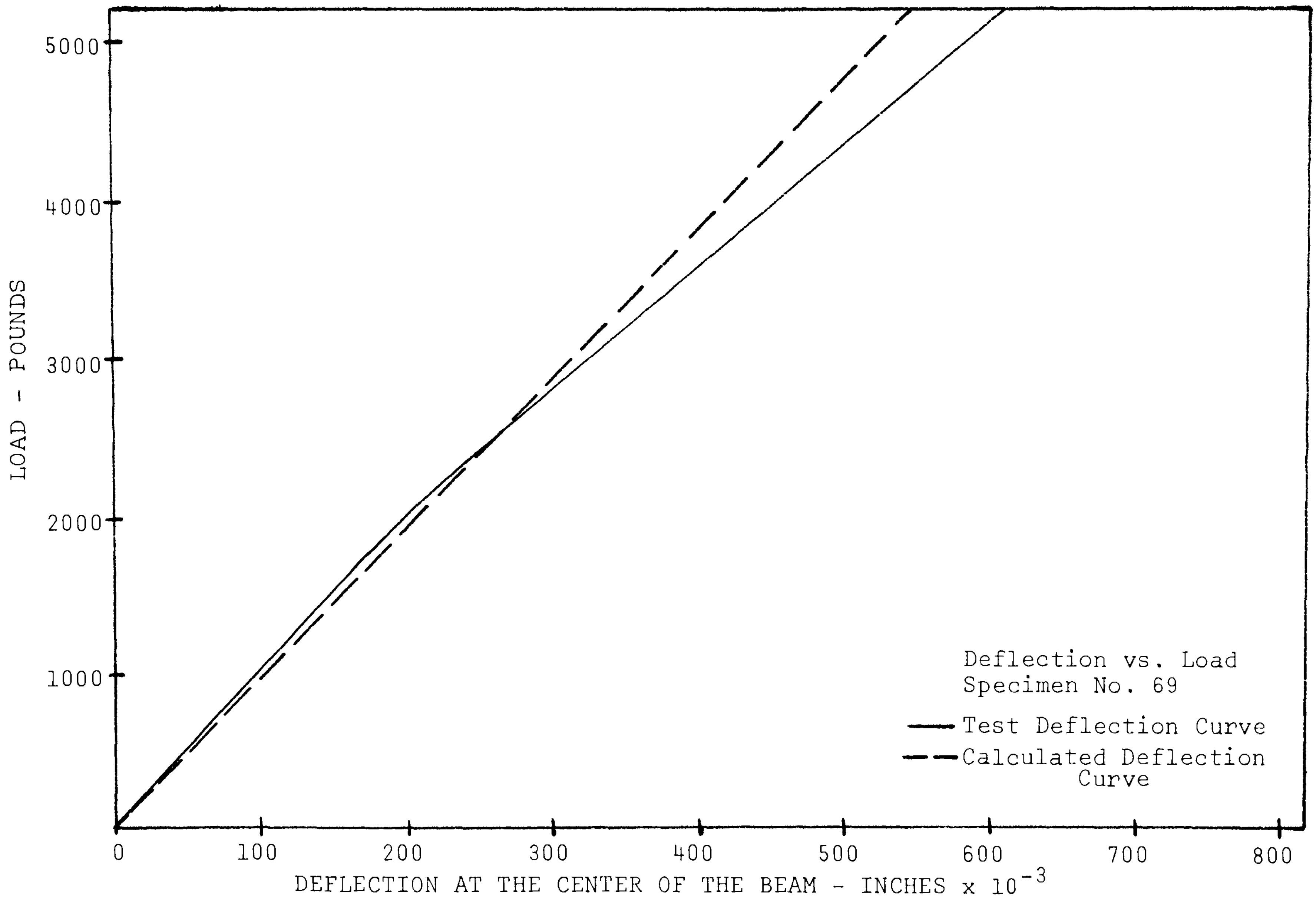


Fig. 6 Deflection vs. Load Curve

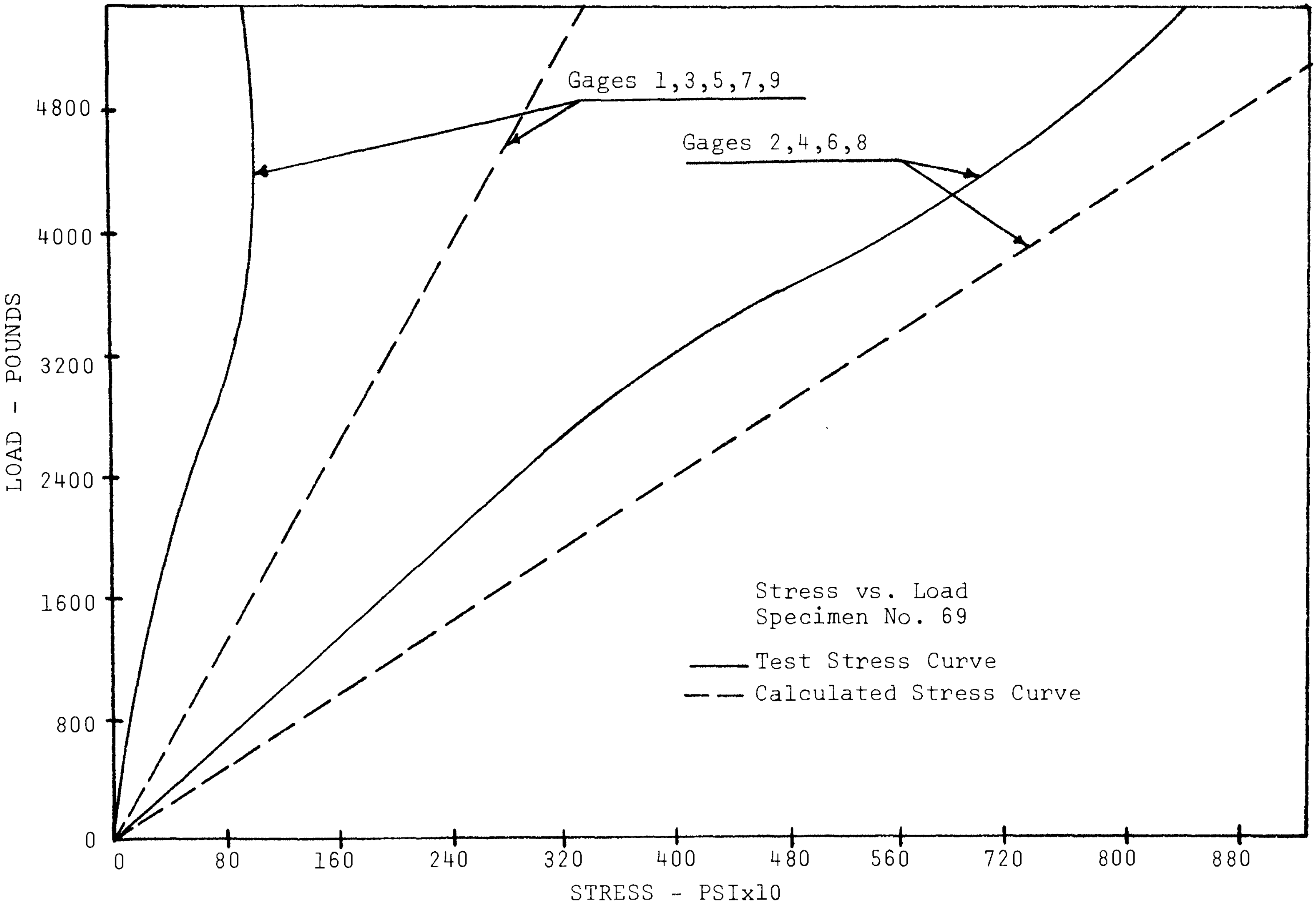


Fig. 7 Stress vs. Load for Strain Gages 1-9

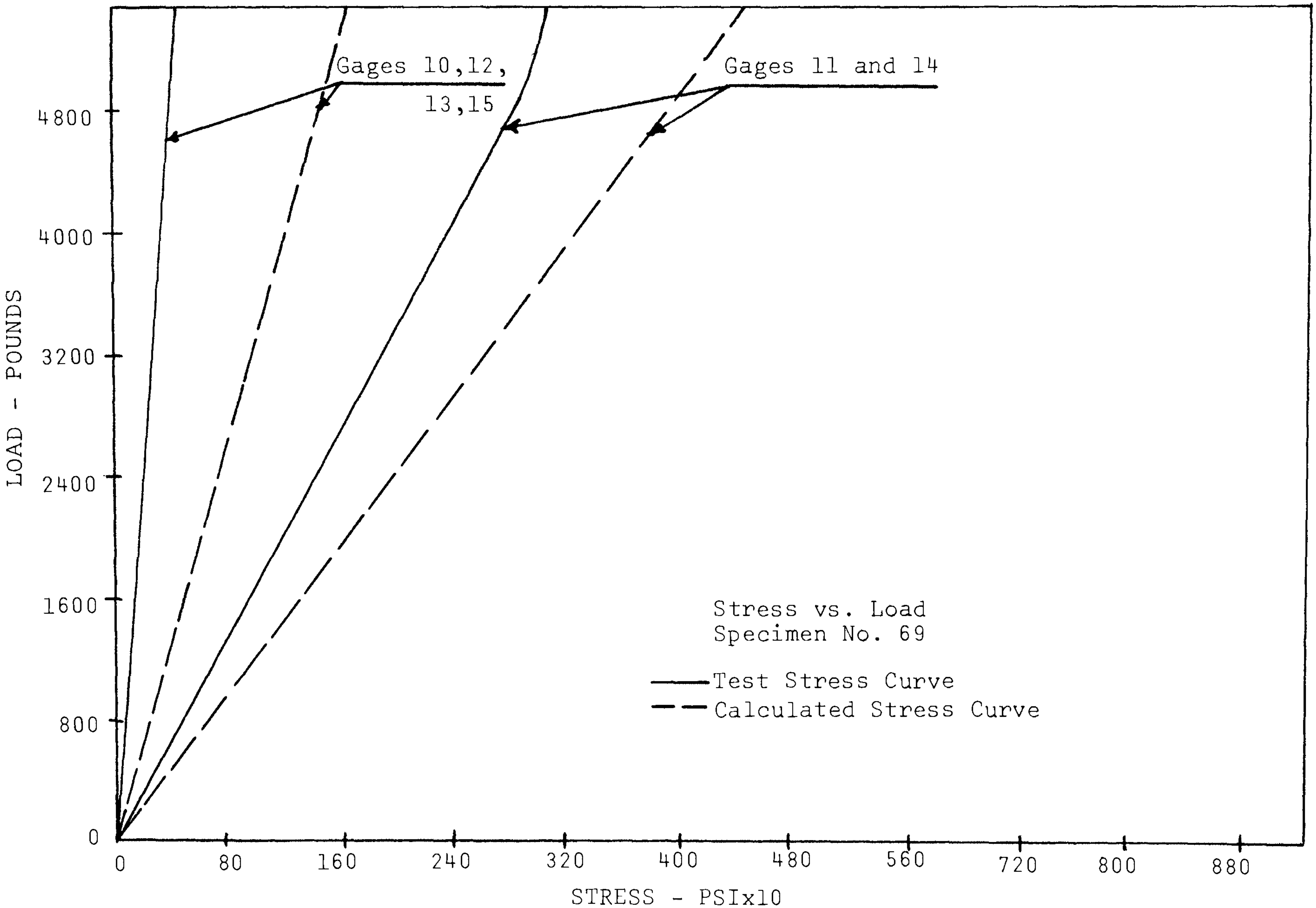


Fig. 8 Stress vs. Load for Strain Gages 10-15

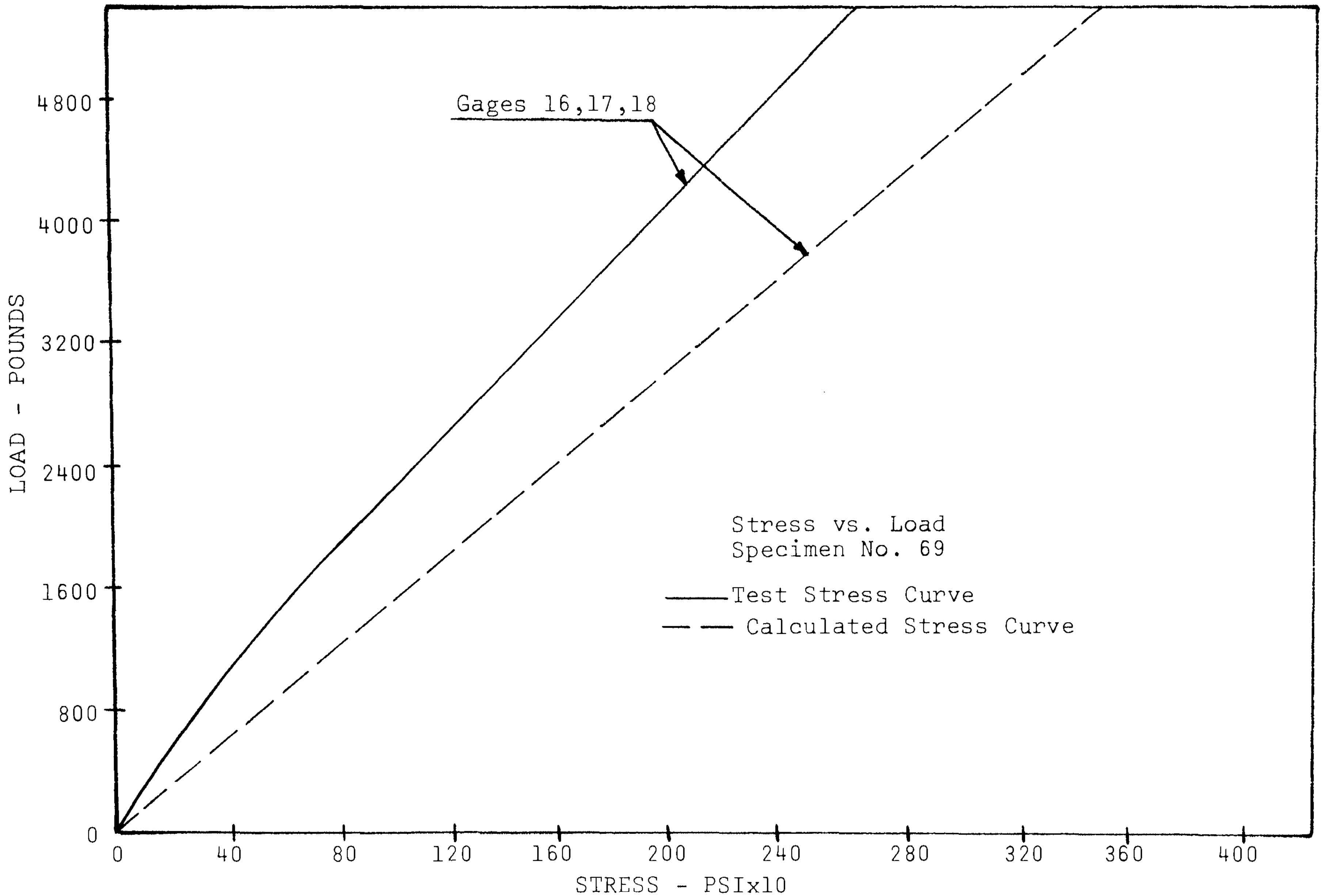


Fig. 9 Stress vs. Load for Strain Gages 16-18

CALCULATIONS FOR SPECIMEN NO. 69

$A_s = 1.026\ in^2$ $\quad E_s = 29{,}000{,}000$ psi $\quad$ Use a 12" width

$w = 110$ lb./ft. $\quad f'_c = 6000$ psi $\quad b = 12$ in.

$d = 2.61$ in.

$$E_c = w^{1.5}\ 33\ f'_c$$

$$E_c = (110)^{1.5}\ 33\ 6000$$

$$E_c = 3{,}955{,}000 \text{ psi}$$

$$n = \frac{E_s}{E_c} = \frac{29{,}000{,}000}{3{,}955{,}000}$$

$$n = 9.834$$

$$p = \frac{A_s}{bd} = \frac{1.026}{12(2.61)}$$

$$p = 0.03276$$

Calculate the k Value

$$k = \sqrt{(pn)^2 + 2pn} - pn$$

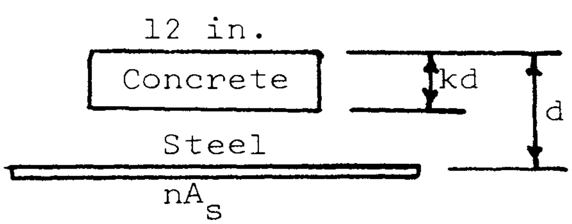
$$k = \sqrt{\left[(0.0328)(9.834)\right]^2 + 2\left[(0.0328)(9.834)\right]} - (0.0328)(9.834)$$

$$k = 0.5427$$

$$kd = 0.5427(2.61) = 1.416\ in^2$$

Calculate the Total Moment of Interia

$$I_s = 0.433\ in^4$$



$$I_t = \frac{bd^3}{3} + n\ A_s((1-k)d)^2 + n\ I_s$$

$$I_t = \frac{12}{3}(1.416)^3 + 9.834(1.026)\ 1.194\ ^2 + 9.834(0.433)$$

$$I_t = 29.976\ in^4$$

Moment Calculation

$$M_r = \frac{f_s\ I_t}{n\ c\ 12}$$

$c = (1-k)d + (t_s - \overline{x})$

$\overline{x}$ = distance from the top of the steel deck to its centroid

t_s = depth of the steel deck

$c = 1.194 + 0.92$

$c = 2.114$ in.

$$M_r = \frac{20,000}{9.834} \times \frac{29.976}{2.114} \times \frac{1}{12}$$

M_r = 2403 ft-lb/12 in.

M_M = 5775 ft-lb/12 in.

M_M = max. test moment

$$\text{Factor of Safety} = \frac{M_M}{M_r}$$

$$= \frac{5775}{2403}$$

$$= 2.403$$

Bond Calculation

$$u = \frac{V}{\Sigma ojd}$$

Σo = 16.375 in.

$$j = 1 - k/3 = 1 - \frac{0.5427}{3}$$

$$j = 0.8521$$

$$jd = 0.8521(2.61) = 2.224 \text{ in.}$$

$$V = \text{max. test load}/2$$

$$V = 3917.5 \text{ lb.}$$

$$u = \frac{3917.5}{16.375(2.224)}$$

$$u = 107.6 \text{ psi} < 350 \text{ psi} \quad \text{OK}$$

Shear Calculation

$$v = V/bd$$

$$V = 3917.516$$

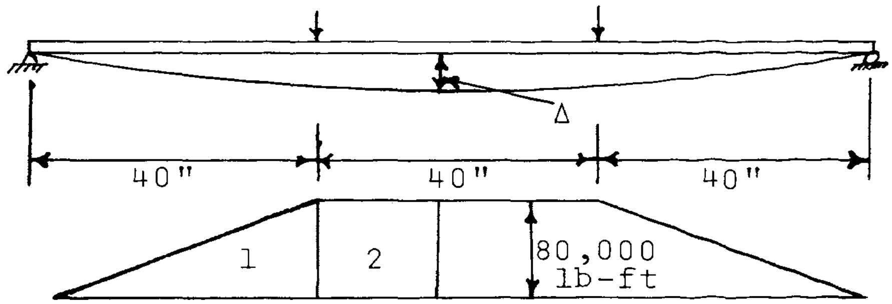
$$v = 3917.5/30\text{x}2.61$$

$$v = 150.0 \text{ psi} < 1.1\sqrt{f'_c} = 85.1$$

OK

Deflection Calculation

Calculate the Deflection at p = 4000 lb.



Moment Diagram

Moment Area Method for Deflection

Area 1	80,000 x 40 x 1/2 x 2/3 x 40	= 42,666,660
Area 2	80,000 x 20 x 50	= 80,000,000
		122,666,666

$$E_c I_t \Delta = \text{Moment of Moment Diagram}$$

$$\Delta = \frac{122{,}666{,}666}{3{,}955{,}000 \times 29.976}$$

$$\Delta = 1.038 \text{ in.}$$

Δ = deflection over a 12 in width

Δ_{30} = deflection over the 30 in test width

$$\Delta_{30} = \frac{1.038}{2.5} = 0.415 \text{ in.}$$

This value is plotted on the load-deflection curve.

Stress Calculations

These stress values are plotted on the load-stress curves for the appropriate gage location.

For these calculations, a 3000 lb. load was chosen which gave a moment of M = 61,381 lb-in.

1. Gages 1,3,5,7,9

$$f_1 = \frac{Mc_1}{I_t}$$

$$c_1 = 0.584 \text{ in.}$$

$$f = \frac{61,381(0.584)}{29.926}$$

$$f = 1198 \text{ psi}$$

2. Gages 2,4,6,8

$$f_2 = \frac{Mc_2}{I_t}$$

$$c_2 = 2.114$$

$$f_2 = \frac{61,381(2.114)}{29.926}$$

$$f_2 = 4433 \text{ psi}$$

3. Gages 16,17,18

$$f_3 = \frac{Mc_3}{I_t}$$

$$c_3 = 1.416$$

$$f_3 = \frac{61,381(1.416)}{29.926}$$

$$f_3 = 2900 \text{ psi}$$

4. Gages 10,12,13,15

$$M = 30{,}000 \text{ lb-in}$$

$$f_4 = \frac{M_{c_4}}{I_t}$$

$$c_4 = 0.584 \text{ in.}$$

$$f_4 = \frac{30{,}000(0.584)}{29.926}$$

$$f_4 = 585 \text{ psi}$$

5. Gages 11,14

$$M = 30{,}000 \text{ lb-in}$$

$$f_5 = \frac{M_{c_5}}{I_t}$$

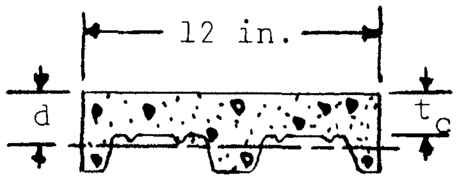
$$c_5 = 2.114$$

$$f_5 = \frac{30{,}000(2.114)}{29.926}$$

$$f_5 = 2120 \text{ psi}$$

APPENDIX B

Physical Properties of Steel Deck



Physical Properties of Specimens

SPECIMEN NO.	AREA in.^{2} 12 in, WIDTH	GAGE	t_c in.	I_s-in.^{4}/ 12 in.	d-in.	f'_c-psi	f_s-psi
66	0.616	20	2	0.260	2.61	8400	20,000
10	0.616	20	2	0.260	2.61	4000	20,000
9	0.616	20	2	0.260	2.61	4000	20,000
69	1.026	16	2	0.433	2.61	6000	20,000
6	0.616	20	3	0.260	3.61	4300	20,000
1	0.616	20	3	0.260	3.61	4300	20,000
68	1.026	16	4	0.433	4.61	6000	20,000

f'_c = Ultimate Concrete Stress f_s = Allowable Steel Stress

Table V Physical Properties of Steel Deck

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IX. VITA

Charles Sherman Bach, Jr. was born December 7, 1946, in St. Louis, Missouri. He received his primary and secondary education in Mehlville, Missouri. He received his college education from the University of Missouri at Rolla. He received a Bachelor of Science degree in Civil Engineering from the University of Missouri at Rolla on August 3, 1968 in Rolla, Missouri.

He has been enrolled in the graduate school at the University of Missouri at Rolla since September, 1968 and has held a research assistantship under Professor J. W. Hubler since entering graduate school.

He is presently registered as an Engineer-in-training with the Missouri State Board of Registration for Architects and Engineers. He has a reserve commission in the United States Army as a Second Lieutenant in the Corps of Engineers.