Curved girder bridge model analysis and testing

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Analyse et essai d'un modèle de pont courbe

Modellberechnung und Versuchsbelastung einer gekrümmten Stahlträgerbrücke

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SUMMARY

A geometric scale model of two span continuous horizontally curved I-girder bridge was analysed and tested. Loadings included steel framing, wet concrete, ten truck locations, and forty-eight unit load positions. A general three dimensional method of analysis and computer program developed at Syracuse University were used. Analytical and experimental bending moments and axial forces were in close agreement.

RÉSUMÉ

Le modèle d'un pont courbe horizontal à deux travées continues à poutres en I a fait l'objet d'essais. Les charges comprenaient l'ossature métallique, le béton, dix cas de charge avec des camions, et quarante-huit positions de charges unitaires. Une méthode générale d'analyse tridimensionnelle et un programme d'ordinateur développé à l'Université de Syracuse ont été employés. La comparaison calcul-expérience pour les moments de flexion et les forces axiales montre une bonne concordance.

ZUSAMMENFASSUNG

Ein Modell einer zweifeldigen, durchlaufenden, horizontal gekrümmten Stahlträgerbrücke wurde untersucht. Belastungsfälle waren die Stahlkonstruktion selbst, das Betongewicht, zehn Lastwagenpositonen und achtundvierzig Stellungen einer konzentrierten Einheitslast. Ein dreidimensionales Berechnungsverfahren und ein Computer-Programm wurden in der Universität von Syracuse entwickelt. Analytische und versuchstechnisch ermittelte Momente und Normalkräfte zeigten gute Übereinstimmung.

1. INTRODUCTION

The Seekonk River Bridge near Providence, Rhode Island contains a two span off ramp designated as spans 7 and 17. This portion of the bridge is horizontally curved and structurally continuous. The structural frame consists of three main I-section welded girders interconnected with full depth X-braced interior diaphragms. One end of the ramp bridge is skewed.

A small scale structure of this bridge (Spans 7 and 17) was constructed, instrumented, and tested, and the experimental results compared to analytical values obtained using a three dimensional method of analysis and computer program [1] developed at Syracuse University. The overall objectives of this study were to demonstrate that meaningful results could be obtained from a small scale geometric model, and to verify the three dimensional method of analysis and computer program.

2. DESIGN OF THE SMALL SCALE HORIZONTALLY CURVED GIRDER BRIDGE

A model that is geometrically scaled to a prototype can be used to study more characteristics than one that is geometrically distorted, thus the first criterion was to develop a structural model that would adhere, through similitude relations, as reasonably as possible to the design structure. The properties of the materials selected for the model have an important role in the development of these similitude relations

Since the model structure was to be tested into the post-yield range, the stress-strain characteristics of the materials in the small scale structure should duplicate those of the prototype structure. Also, the use of materials that are similar in physical characteristics for stressed members of the model structure and those of the prototype structure produces more consistent and reliable data than can be achieved with dissimilar materials. This is particularly significant when a considerable number of readings need to be taken with static loading conditions that must be maintained constant for a period of time. The same grades of steel used for the major structural steel members of the prototype were thus selected for the model - A441 for the flanges of the girders and A36 for all other structural members.

One of the most important considerations in the selection of the scale factor was the welding of components, particularly welding the flanges to the web and the stiffeners to the web. Distortion of the thin web can be minimized by the use of a low temperature eutectic welding rod, however, too thin a web will still distort even with all possible precautions. Thus the minimum thickness web to prevent excessive distortion was a major selection, and for this model the web thickness was chosen as 1.9mm. Since the prototype web thickness was 12.7mm, a scale reduction of 1:6.667 results. This is the ratio of the linear dimensions of the model to those of the prototype.

The concrete of the deck of the prototype was duplicated as nearly as possible in the small scale structure. In general, stiffness is the most significant characteristic of the deck and similarity of this property can be obtained with considerable accuracy.

The steel framing for the small scale structure was designed at a scale reduction factor of 1:6.667. Thus n, the geometric scale of the prototype to the model, L_p/L_m, is 6.667. Since the prototype bridge framing is A36 and A441 steel and the small scale laboratory structure is of the same steels the resulting geometric scale n, based on modulii of elasticity, would be 1.00. This would require the small scale structure and the prototype to be the exact same size. Adjustments to the selected expressions are thus necessary and one factor in each relation associated with the modulus of elasticity - E, must compensate. To maintain geometric similarity, L and I should remain uncompensated--thus force F_m, is the function to be adjusted.

Rewriting the expression for Bending Deflection ($\Delta = \frac{FL^3}{EI}$), recognizing that $E_m = E_p$, and maintaining a scale factor for $\Delta_p : \Delta_m$ of n

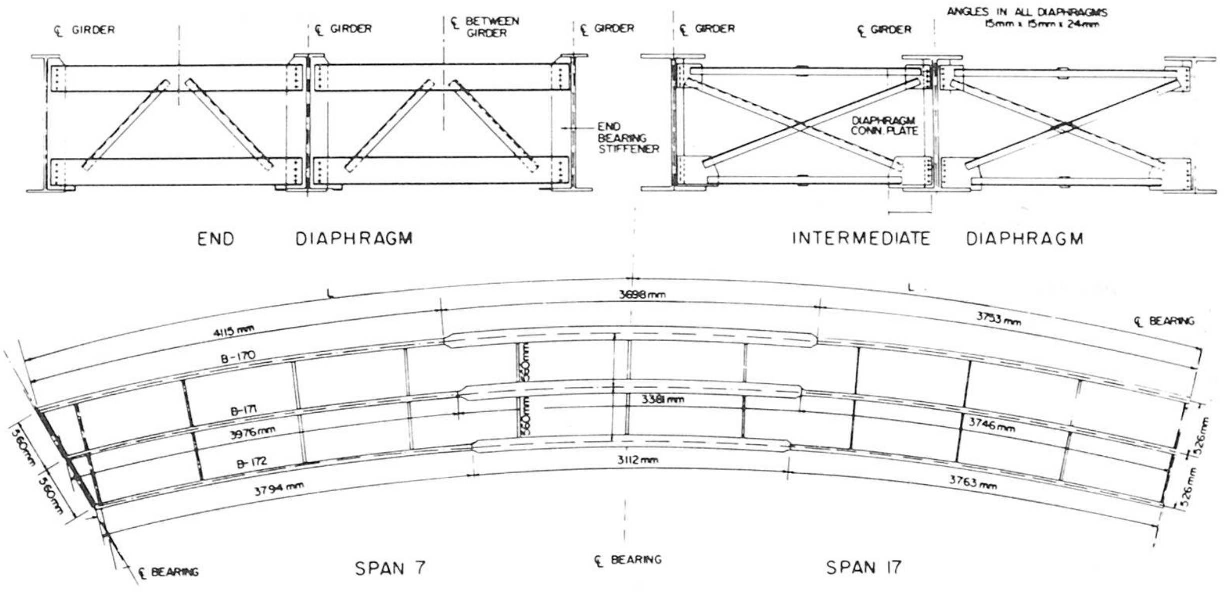
$$\frac{\Delta_p}{\Delta_m} = n = \frac{\dfrac{F_p L_p^3}{E_p I_p}}{\dfrac{F_m L_m^3}{E_m I_m}} = \frac{F_p}{F_m} = \frac{E_m}{E_p} = \frac{L_p^3}{L_m^3} = \frac{I_m}{I_p} = \frac{F_p}{F_m} = \frac{n^3 L_m^3}{L_m^3} = \frac{I_m}{n^4 I_m} = \frac{1}{n}\frac{F_p}{F_m}$$

$$F_m = \frac{F_p}{n^2}$$

In a similar manner, general scale factors were derived for the other quantities listed in Table 1.

3. CONSTRUCTION OF THE MODEL

The construction of the small scale structure patterned after Spans 7 and 17 of the Seekonk River Bridge used A441 steel, A36 steel and Portland cement concrete as the principal materials. The steel framing plan of the small scale structure is shown in Figure 1.



GIRDER FLANGE CROSS SECTIONS

GIRDER	FLANGE	LEFT mm	CENTER mm	RIGHT mm
B-170	TOP	80 x 3.81	145 x 7.62	61 x 3.81
	BOTTOM	107 x 5.72	145 x 7.62	88 x 5.72
B-171	TOP	57 x 3.81	137 x 7.62	57 x 3.33
	BOTTOM	84 x 5.72	137 x 7.62	57 x 3.33
B-172	TOP	57 x 2.87	126 x 6.65	57 x 2.87
	BOTTOM	80 x 4.78	126 x 6.65	80 x 4.78

WEBS OF ALL GIRDERS 2.74 mm x 1.91 mm

FRAMING DIMENSIONS

SPAN	GIRDER	RADIUS mm	LENGTH C TO C BEARING mm
7	B-170	23498	5933
	B-171	22741	5626
	B-172	21984	5321
17	B-170	20958	5631
	B-171	22741	5476
	B-172	21984	5349

Fig. 1 Framing Plan of Small Scale Seekonk River Bridge

TABLE 1 SEEKONK RIVER SMALL SCALE STEEL CURVED GIRDER BRIDGE
SIMILITUDE RELATIONS - 6.667 SCALE

	General Scale	Small Structure Scale
Length	$L_m = \frac{L_p}{n}$	$L_m = \frac{L_p}{6.667}$
Area	$A_m = \frac{A_p}{n^2}$	$A_m = \frac{A_p}{44.444}$
Moment of Inertia	$I_m = \frac{I_p}{n^4}$	$I_m = \frac{I_p}{1975}$
Acceleration	$a_m = a_p$	$a_m = a_p$
Time	$T_m = \frac{T_p}{\sqrt{n}}$	$T_m = \frac{T_p}{2.582}$
Frequency	$N_m = N_p\sqrt{n}$	$N_m = 2.582\ N_p$
Velocity	$V_m = \frac{V_p}{\sqrt{n}}$	$V_m = \frac{V_p}{2.582}$
Mass	$m_m = \frac{W_p}{n^2}$	$m_m = \frac{W_p}{44.444}$
Weight - concentrated	$W_m = \frac{W_p}{n^2}$	$W_m = \frac{W_p}{44.444}$
Weight - distributed linearly	$w_m = \frac{w_p}{n}$	$w_m = \frac{w_p}{6.667}$
Weight - distributed over area	$w_m = w_p$	$w_m = w_p$
Force	$F_m = \frac{F_p}{n^2}$	$F_m = \frac{F_p}{44.444}$
Stress	$s_m = s_p$	$s_m = s_p$
Axial Deformation	$\Delta_m = \frac{\Delta_p}{n}$	$\Delta_m = \frac{\Delta_p}{6.667}$
Bending Deflection	$\Delta_m = \frac{\Delta_p}{n}$	$\Delta_m = \frac{\Delta_p}{6.667}$
Torsional Rotation	$\theta_m = \theta_p$	$\theta_m = \theta_p$

n - scale reduction factor
m,p - subscripts denoting the model and the prototype respectively

The girder flanges were A441 steel and the girder webs and all other steel members and portions of members were A36 steel. Bolts for the model were high strength Allen screws - SAE 6-40 (3.5mm diameter) to represent the 22.2mm diameter diaphragm connector bolts of the prototype and 4mm diameter for the 25.4mm splice bolts of the prototype. Portland cement concrete for the deck was mixed in the laboratory.

The steel for the flanges of the girders (A441) was obtained in the stock thickness just greater than the thickness required. These initial stock thicknesses were 6.35mm and 7.94mm. The steel for the webs of the girders and all other members and sub members was A36. All components of the small scale structure fabricated from steel plate were thus ground to the required thickness. Following the grinding of the A441 steel plates to the required thicknesses, each flange plate was scribed to proper radius and width using offsets from each semi-chord of each flange segment, and then rough cut.

To mill the edges of the flanges to the prescribed radii, a hydraulic tracing machine was attached to the milling machine. A template was fit to the scribed arc on the plate and clamped to the plate. The hydraulic tracing stylus followed the curvature of the template and the milling cutter, receiving the signal from the tracer, machined the flange edge to the scribed arc. Great accuracy is obtained with this technique.

The webs of the girders have cambers of both positive and negative curvature. These were also laid out by offsets from a base line. The edges were milled to the finished curvature by the same techniques used to edge mill the flanges.

Other members, including angles and channels for the diaphragms, were milled or ground to thickness and flanged. Stiffeners and splice plates were ground and end milled to size.

Alignment blocks were used in the assembly of the flanges, webs and intermediate stiffeners. Then alignment blocks were machined to the proper height and width and were used to accurately locate the relative positions of the web and the flanges and to accurately position the stiffeners. With the alignment blocks in place the girder components were tack welded. The blocks were then removed and final welding completed.

The girders and diaphragms were assembled by bolting. Predrilled holes in the diaphragm plates and gusset plates permitted accurate location of these members. The angle members were placed in their proper position in the diaphragm and clamped to the gusset plates. The diaphragm assemblage was removed from the structure, welded, replaced between the girders, and bolted into place.

An interruption in construction was necessary to permit gravity load testing for the structural frame dead load and for the wet concrete dead load. After these tests were completed steel shear studs were welded to the top flange of each girder using a Nelson Studweld gun. Forms were then placed and a slab 34.3mm thick representing the 229mm slab of the prototype were poured and cured.

The deck was made from Type 3 Portland cement, gravel, sand and water. A mix by weight of 1 part cement, 2 parts sand, 2 parts gravel and a water-cement ratio of 0.52 by weight, produces concrete with an ultimate strength in excess of 5,000 psi and a modulus of elasticity exceeding 4,000,000 psi. The gravel used had a maximum size of about 5mm. Hardware cloth was used for the reinforcing. A section through this deck is shown in Figure 2.

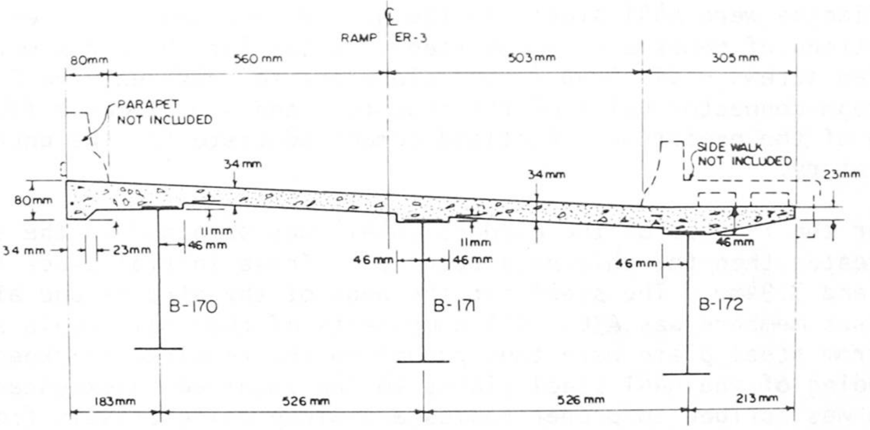


Fig. 2 Section Through Small Scale Bridge

4. THE EXPERIMENTAL PROGRAM FOR THE SEEKONK RIVER BRIDGE SMALL SCALE STRUCTURE

The small scale bridge structure described in the previous section was assembled on concrete piers. The supports at one end of the bridge (at the radial diaphragm) and at the center pier consisted of one 44 448N capacity BLH type C, SR-4 load cell under each girder, a total of 6 load cells for these locations. At the other end of the bridge (at the skew diaphragm) a steel ball 38mm in diameter was placed under each girder. The ends of the bridge were held down to the test floor.

Loads for stress and deformation conditions below the proportional limit were applied to the structure through gravity. Several hundred containers weighing 53.4N, bags containing shot and punchings filled to specified amounts, and large accurately weighed steel plates were used for distributed loadings as shown in Figure 3.

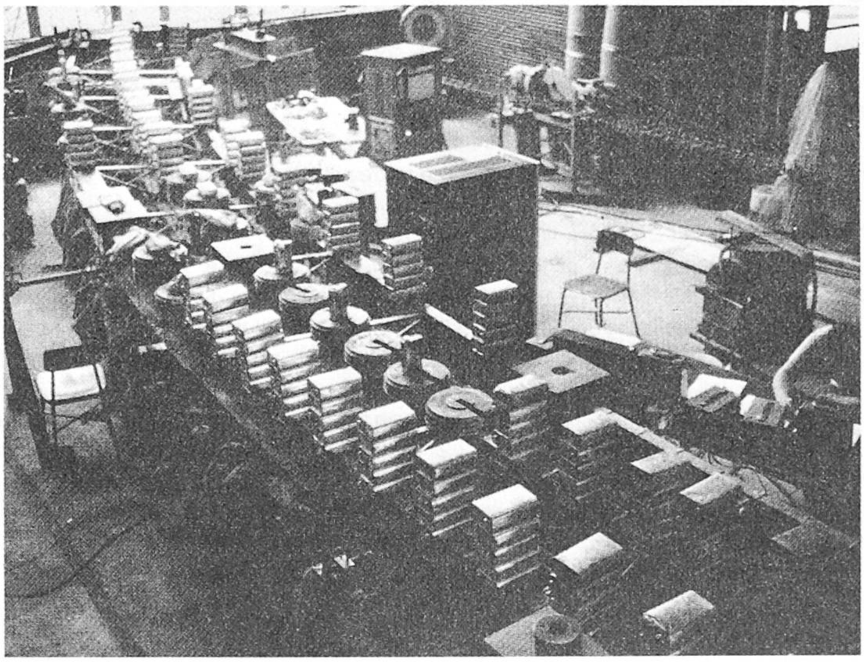


Fig. 3 Dead Loading on Small Scale Seekonk River Bridge

For concentrated load tests, frames were made to accurately position 8.896N at the load points. A frame that straddled the members was used for the steel framing without deck and a cantilever frame was used for the structure with deck. These frames were moved and positioned using an overhead crane.

The locations of strain gages were selected to provide the maximum information possible from the tests. A total of 241 strain gages were attached to the small scale bridge. These gages were BLH, SR-4 foil gages with a polyimide base (FAE-12-12S6). They were cemented to the member using BLH epoxy cement EPY-150. To measure deflections, sensitive dial indicators were located to read vertical movements at each end of the bridge at each girder, at the center pier of the bridge at each girder and at the center of each span at each girder.

Sixty-four independent sets of static loadings were applied to the small scale bridge. These consisted of steel framing dead load (Figure 4); wet concrete dead load on Span 7, on Span 17, and on both spans; wet concrete on both spans with modifications; wet concrete on the outer portions of both spans followed by the addition of the center area (Figure 5); 24 sequentially applied concentrated loads at joints without the deck in place, and 24 with the deck; and 10 live loading positions as shown in Figure 6.

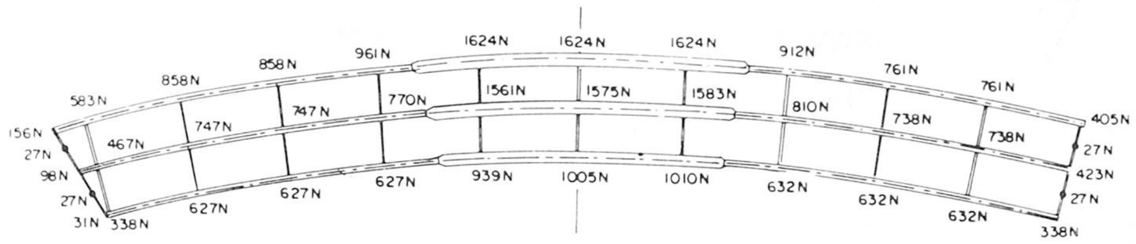


Fig. 4 Steel Framing Loads Transfered to Mathematical Joints

As noted in Table 1, the distributed dead load of the prototype girder should be divided by the scale factor to obtain the model girder dead load. However, because necessary adjustment was made to the similitude relations, the weight per foot of the model is the distributed weight of the prototype girder divided by the scale factor squared. This results in a weight deficiency for the steel framing loads and stress conditions and deflections determined. This external loading is shown in Figure 4.

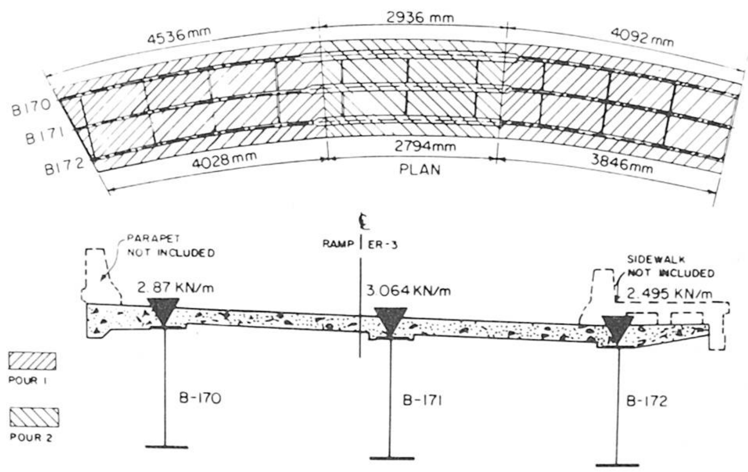
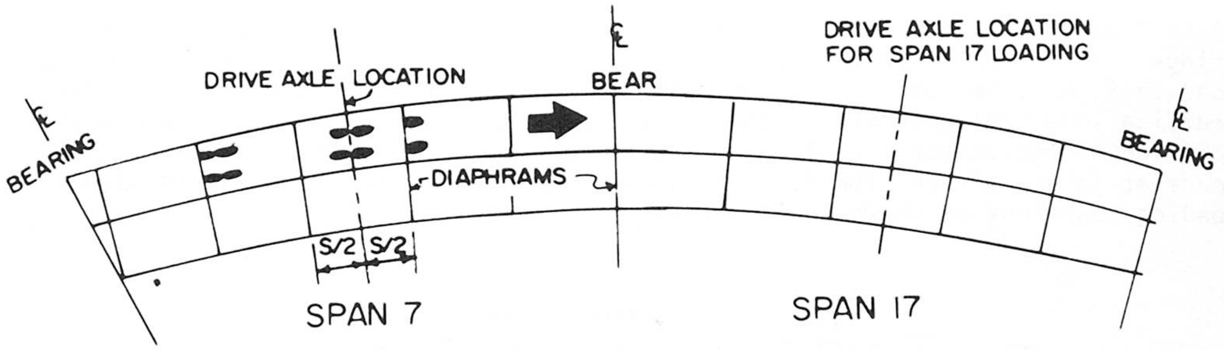


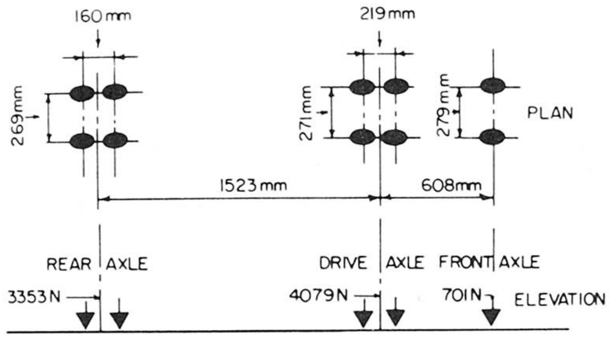
Fig. 5 Wet Concrete Loading Sequence as Poured on Prototpye Structure

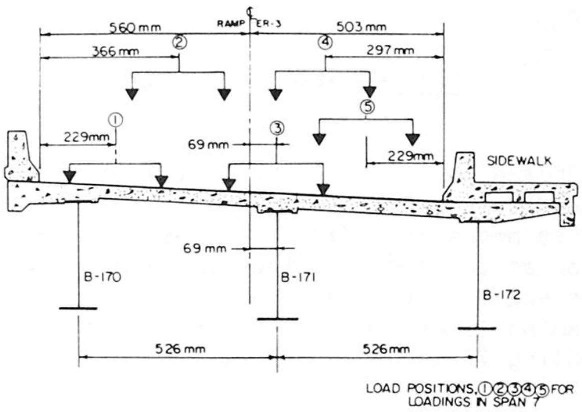
Among the set concrete tests performed was a set based on concrete loading data supplied from the field after the prototype slab was poured. The sequence used in placing the concrete in the field was used on the small scale structure. Both the scaled intensity of loads, and the sequence of loading are shown in Figure 5. Because of the magnitude of these loads, and because the structural action was previously observed to be linear, one-half of the concrete loading shown in Figure 5 was applied to the girders and the results were doubled.

Live loadings were applied to the small scale structure sequentially as shown in Figure 6. The wheel spacings and intensities of loading were scaled to the test vehicle of the Federal Highway Administration. This vehicle had been used to test the prototype structure.



Circumferential Location of Live Load





Static Live Load Forces and Dimensions Transverse Locations of Live Loads

Fig. 6 Live Load Intensities and Locations on Small Scale Structure

Early loadings of the bridge model were done in increments of 1/2 of the total load to be added. This means that for each loading condition strain and deflection readings were taken at: (a) 0 load; (b) 1/2 full load; (c) full load; (d) 0 load.

After the linearity of loading was proven this sequence was eliminated and only zero readings were made.

Strain data was the input to a computer program that produced several quantities, including stress. To obtain values of experimental bending moment in the girders, experimental stress values were plotted at the locations where they occurred in the cross sections and a complete stress distribution curve across the section was made. Stress values from this curve were used in a program to compute values of moment.

5. MATHEMATICAL ANALYSIS OF SEEKONK RIVER BRIDGE MODEL

The model was analyzed using a three dimentsional method of analysis [1], [2], [3], developed at Syracuse University. For this analysis, the bridge structure is divided into an appropriate number of members meeting at real or artificial joints and the loads are applied at these joints. The analysis proceeds by expressing the necessary axial forces, shears, bending moments, and torsional moments acting on the ends of each of the members in terms of the three components of translational displacement and three components of rotational displacement that may occur at each end of each member. Next, the joint displacements that are referred to a general coordinate system are broken into components giving the displacements for each member in terms of the member's own local coordinate system. Finally, the equilibrium equations for each joint are satisfied and expressed as functions of the joint displacements. The resulting set of equilibrium equations for the structure is solved for the unknown joint displacements, which in turn are used to compute forces and moments. This method of analysis has been programmed for a digital computer [4]. The general coordinate system and joint locations for the analyses are shown in Figure 7.

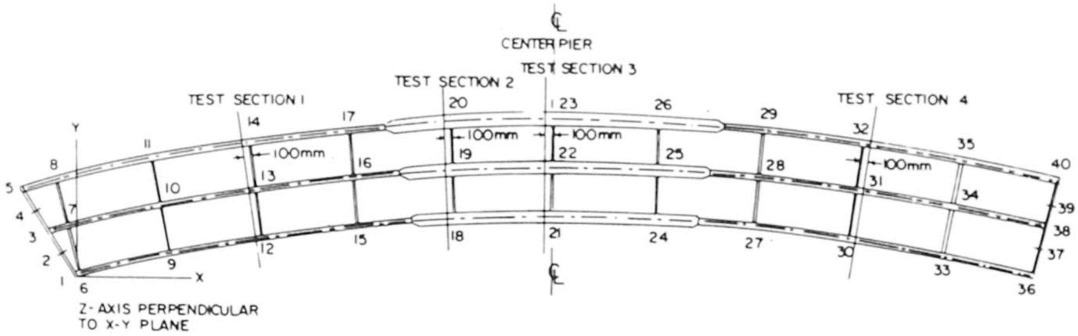


Fig. 7 General Coordinate System and Test Section Locations

6. RESULTS OF ANALYTICAL AND EXPERIMENTAL STUDIES

Comparisons of analytical and experimental values of bending moments in the girders at several instrumented sections (see Figure 7) are given in Tables 2, 3, and 4. Loading cases for the model, both with and without the composite concrete deck, are included. Additional comparisons of both moments and deflections are included in Figures 9, 10, and 11. The correlation between experimental and theoretical results for the girders is excellent for bending moments and good for deflections.

The Seekonk River Bridge is dependent upon the reinforced concrete deck, ten groups of interior diaphragms, and two groups of end diaphragms for stability and transverse load transfer. Typical diaphragms are shown in Figures 1 and 8. For intermediate diaphragms the top and bottom members are back to back angles, and the diagonal members are single angles. All angles are of the same section and for the small scale structure each has an area of 2.87 square millimeters. The end diaphragms consist of channels and angles.

For mathematical analysis, the extended intersection of the diaphragm planes to the girders defined the theoretical joints at the center of the girder webs in a vertical plane. As a consequence each diaphragm member at the girder locations is eccentric to the theoretical joint. These eccentricities are important in the

TABLE 2 Experimental and Analytical Dead Load Bending Moments in Girders

Loading	Test Section	Bending Moments KN-M					
		Girder B-170		Girder B-171		Girder B-172	
		Test	Theory	Test	Theory	Test	Theory
Steel Framing	1	1.69	1.81	1.02	1.24	0.79	0.79
	2	-1.13	-1.13	-1.02	-0.79	-0.68	-0.45
	3	-4.29	-4.52	-3.73	-3.73	-2.6	-2.37
	4	*	1.36	1.02	1.02	0.79	0.79
Concrete Field Pour I	1	6.55	6.89	4.97	4.75	2.71	3.16
	2	-4.86	-5.09	-4.18	-4.29	-3.73	-2.83
	3	-12.77	-13.11	-10.96	-11.41	-8.25	-8.36
	4	4.97	5.31	4.18	4.29	3.39	3.39
Concrete Field Pours 1 & 2	1	6.44	6.56	4.41	4.63	2.94	3.16
	2	-4.97	-4.97	-3.96	-3.96	-2.26	-2.49
	3	-16.16	-15.71	-13.56	-13.67	-10.40	-10.17
	4	4.63	5.88	4.18	4.63	3.50	3.39

TABLE 3 Experimental and Analytical Live Load Bending Moments in Girders

Loading	Test Section	Bending Moments KN-M					
		Girder B-170		Girder B-171		Girder B-172	
		Test	Theory	Test	Theory	Test	Theory
1	1	4.18	4.29	1.81	1.81	-0.23	-0.12
	2	0.34	0.23	-0.23	0.11	-0.11	0.00
	3	-2.60	-2.83	-1.70	-1.36	0.00	-0.11
	4	-0.79	-1.13	-0.45	-0.57	0.00	0.00
2	1	3.50	3.73	2.15	2.03	0.11	0.57
	2	0.45	0.34	-0.11	0.11	0.00	0.00
	3	-2.15	-2.26	-1.81	-1.47	-0.34	-0.45
	4	-1.13	-1.02	-0.45	-0.57	-0.11	-0.11
3	1	2.60	2.49	1.92	2.03	1.24	1.47
	2	0.57	0.57	0.00	0.11	0.00	0.11
	3	-1.36	-1.36	-1.81	-1.36	-1.02	-0.90
	4	-1.13	-0.79	-0.57	-0.57	*	-0.34
4	1	1.58	2.03	2.26	2.15	2.03	2.26
	2	0.45	0.45	0.00	0.23	0.23	0.11
	3	-0.90	-1.02	-1.70	-1.47	-1.47	-1.47
	4	-0.90	-0.68	-0.57	-0.57	-0.57	-0.34
5	1	1.13	1.70	2.03	2.03	2.03	2.03
	2	0.68	0.57	0.45	0.45	*	0.34
	3	-0.45	-0.57	-1.47	-1.36	-1.58	-1.58
	4	-0.68	-0.57	-0.45	-0.45	-0.45	-0.34
6	1	-1.13	-1.02	-0.45	-0.57	0.00	-0.11
	2	-3.05	-2.49	-1.47	-1.13	-0.11	-0.11
	3	-3.05	-3.16	-0.90	-1.47	0.00	-0.11
	4	3.28	3.50	1.58	1.47	-0.11	-0.23

(TABLE 3 Continued)

Loading	Test Section	Bending Moments KN-M					
		Girder B-170		Girder B-171		Girder B-172	
		Test	Theory	Test	Theory	Test	Theory
7	1	-0.90	-1.13	-0.34	-0.45	-0.11	-0.23
	2	-2.26	-2.03	-1.36	-1.13	-0.34	-0.34
	3	-2.26	-2.60	-1.92	-1.47	-0.45	-0.45
	4	2.83	2.94	1.58	1.58	0.34	0.34
8	1	-0.79	-0.79	-0.68	-0.45	-0.23	-0.23
	2	-1.70	-1.36	-1.36	-1.13	-0.90	-0.79
	3	-1.47	-1.36	-1.81	-1.81	-1.24	-1.02
	4	2.03	2.03	1.92	1.58	1.24	1.24
9	1	-0.68	-0.68	-0.68	-0.45	-0.34	-0.34
	2	-1.36	-1.13	-1.36	-1.13	-1.13	-1.02
	3	-1.02	-0.90	-1.58	-1.58	-1.58	-1.47
	4	1.24	1.58	1.58	1.81	1.58	1.70
10	1	-0.68	-0.68	-0.57	-0.45	-0.45	-0.34
	2	-0.90	-0.90	-1.13	-1.13	-1.24	-1.24
	3	-0.57	-0.68	-1.58	-1.58	-1.92	-1.81
	4	1.24	1.36	1.47	1.81	2.03	1.92

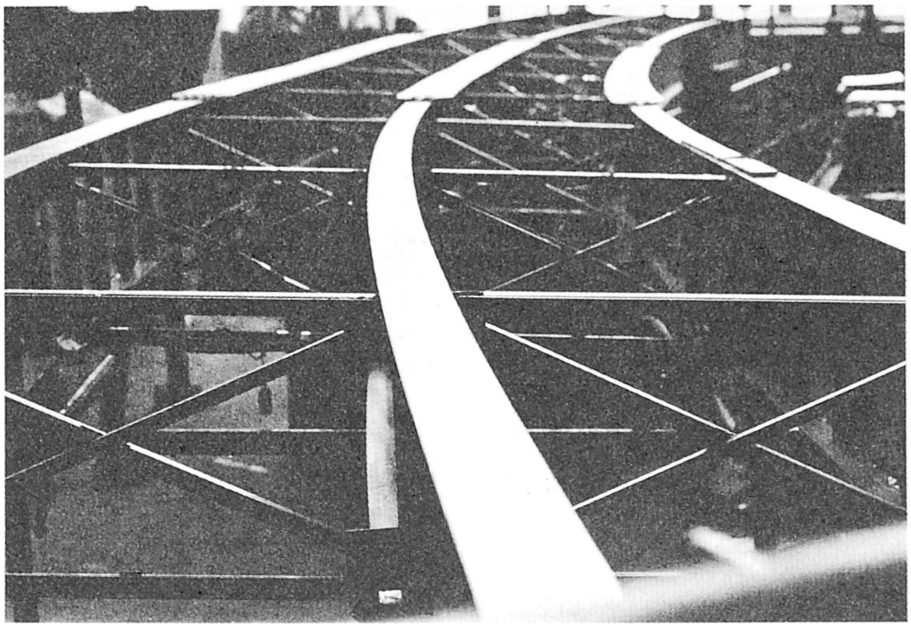


Fig. 8 Intermediate Diaphragms

TABLE 4 CONCENTRATED LOAD BENDING MOMENTS

Test Section	8.90 KN Load at Joint	Girder B-170 Test	Girder B-170 Theory	Girder B-171 Test	Girder B-171 Theory	Girder B-172 Test	Girder B-172 Theory
			Structure Without Concrete Deck				
	15	*	0.226	1.81	2.03	3.16	3.05
1	16	2.6	2.6	1.24	1.36	1.02	1.24
	17	4.75	4.52	1.36	1.58	-1.24	-0.904
	15	0.68	0.79	-1.24	-1.47	-3.95	-3.95
3	16	-1.36	-1.69	-2.6	-2.37	-1.36	-1.36
	17	-4.86	-4.75	-1.47	-1.69	0.56	0.56
	30	0.90	0.90	-1.58	-1.92	-3.95	3.95
3	31	-1.92	-2.15	-2.15	-2.03	-1.58	-1.58
	32	-5.2	-5.2	-2.03	-2.26	0.678	0.794
	30	-0.34	-0.23	1.69	1.92	3.05	3.05
4	31	2.82	3.39	3.84	3.62	1.36	1.81
	32	8.02	8.02	1.69	2.37	-1.58	-1.47
			Structure With Concrete Deck				
	15	0.56	0.45	1.92	2.03	3.16	2.94
1	16	2.49	2.49	1.47	1.47	0.904	1.24
	17	4.63	4.07	1.69	1.81	-0.678	-0.452
	15	0.339	0.45	-0.90	-1.36	-3.28	-3.39
3	16	-1.81	-1.69	-1.92	-2.15	-1.24	-1.24
	17	-4.29	-4.18	-1.92	-1.58	0.339	0.339
	30	0.45	0.226	-1.36	-1.58	-2.6	-3.05
3	31	-2.03	-1.92	-2.03	-1.58	-1.24	-1.36
	32	-3.62	-4.18	-1.58	-1.69	0.226	0.339
	30	0	0.45	2.03	2.37	5.65	5.76
4	31	2.82	3.28	3.5	3.84	1.58	2.03
	32	6.67	7.68	1.81	2.49	-0.90	-0.79

study of diaphragm action.

The forces in all diaphragm members were determined analytically using the three dimensional analysis. All diaphragm members were assumed to carry only axial force without bending. The connections of diaphragm members are therefore assumed pinned. Because of the large slenderness ratios and low torsional resistance of these members, this is valid.

To assist in interpreting diaphragm action, influence surfaces for axial force in the diaphragm members between Joints 15, 16, and 17 of the small scale structure with deck were drawn (see Figure 11 for one of these influence surfaces). These influence surfaces show that at this sensitive section (a) the forces in the diaphragm members are less than 16 percent of the concentrated load applied to the deck at any location (most bar forces are less than 10 percent), and (b) loads in the adjoining span have very small effect on any diaphragm member, the greatest effect being about 6 percent.

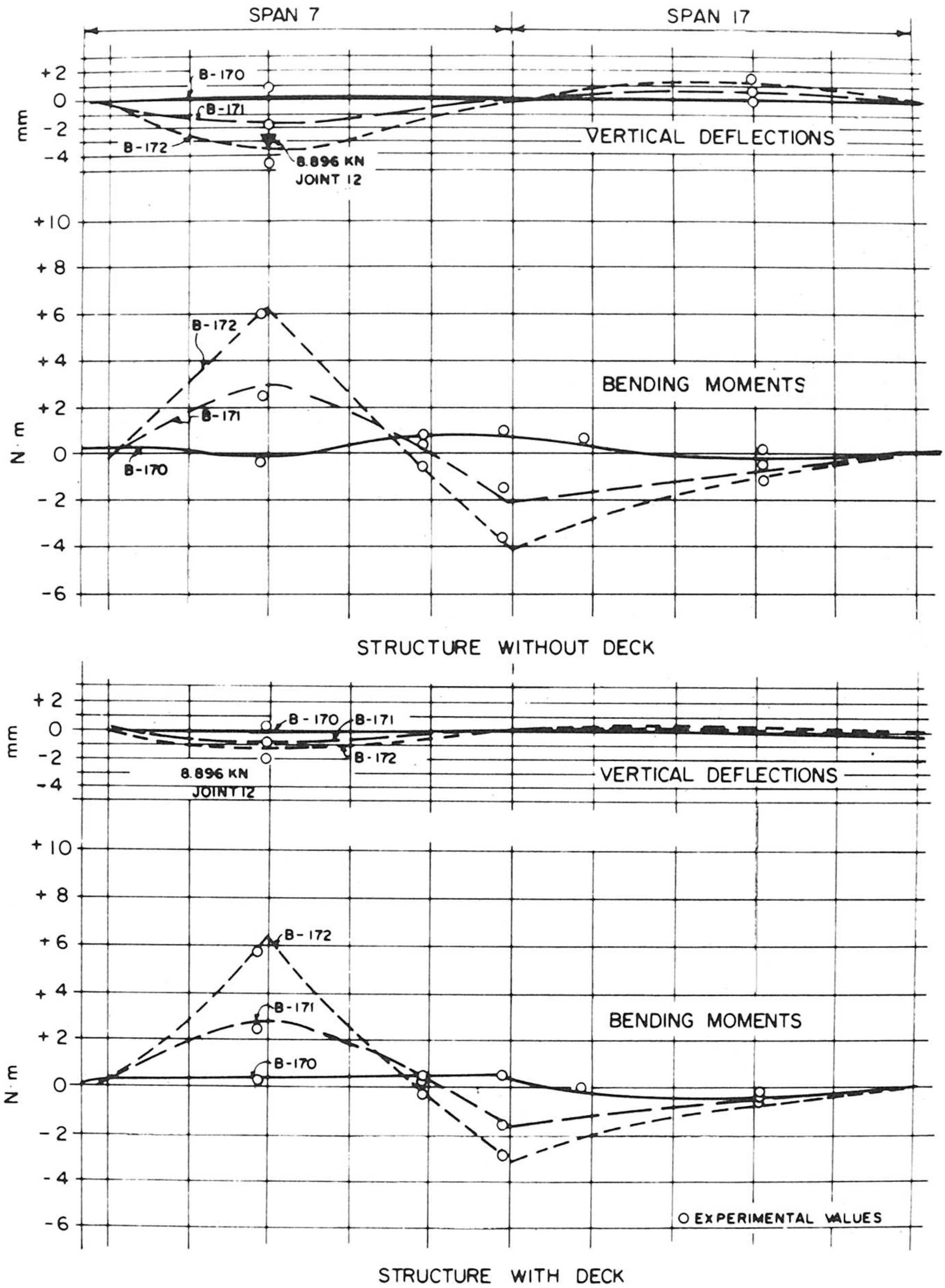


Fig. 9 Analytical and Experimental Moments and Deflections for Load at Joint 12
Small Scale Structure without Deck and with Deck

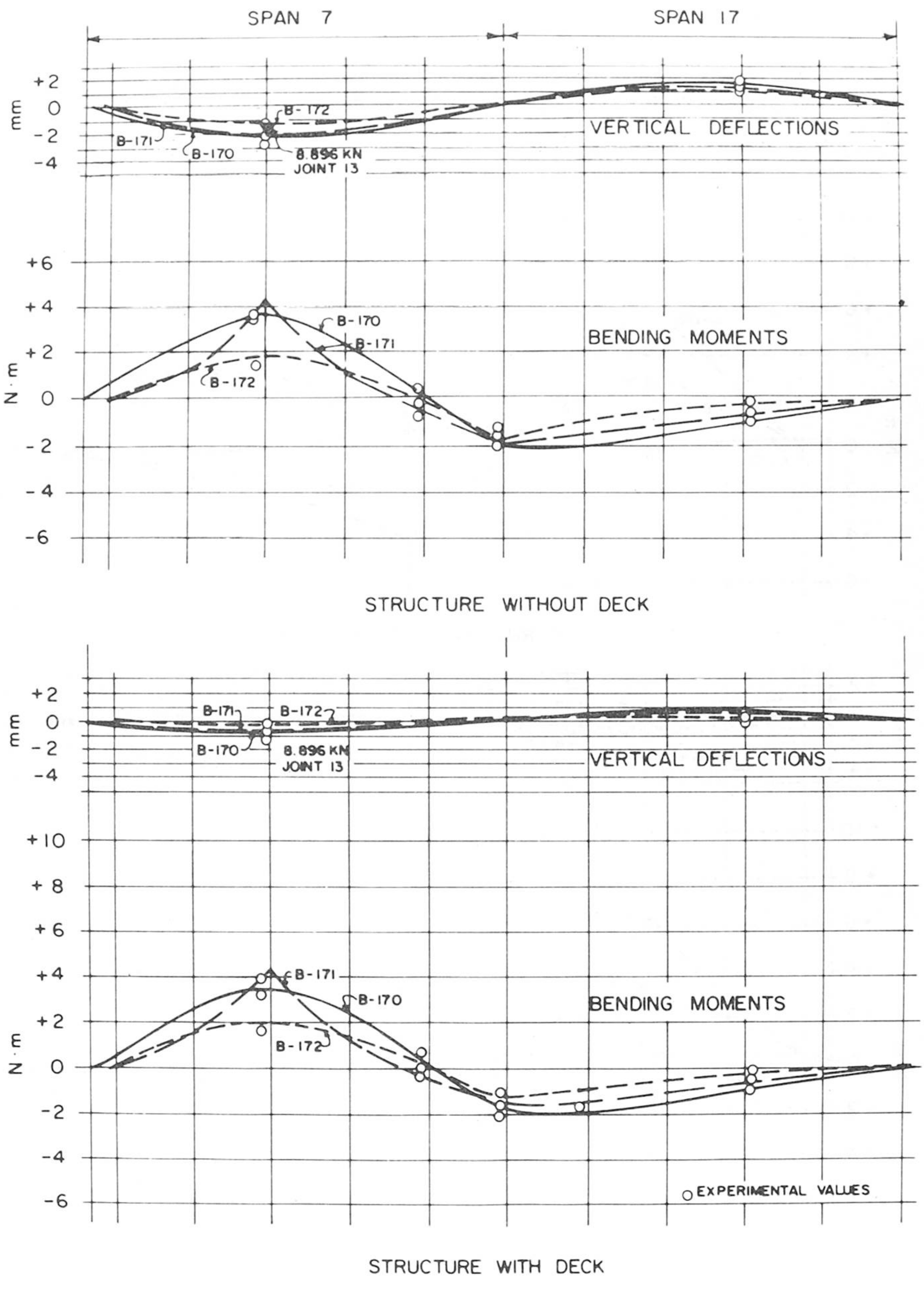
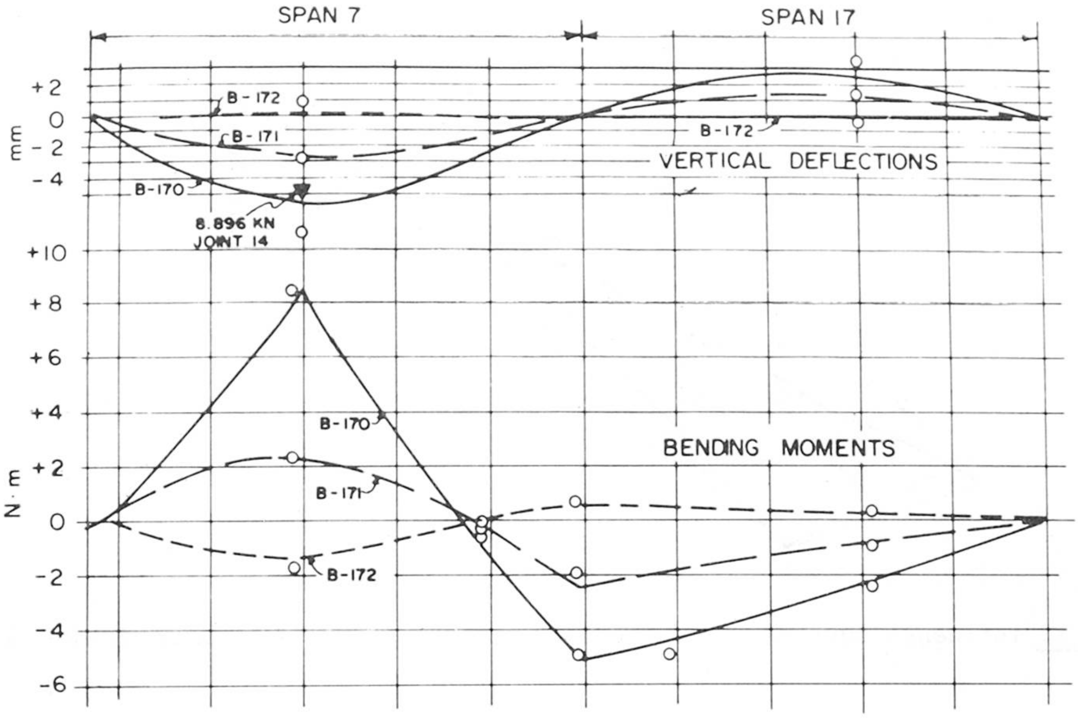
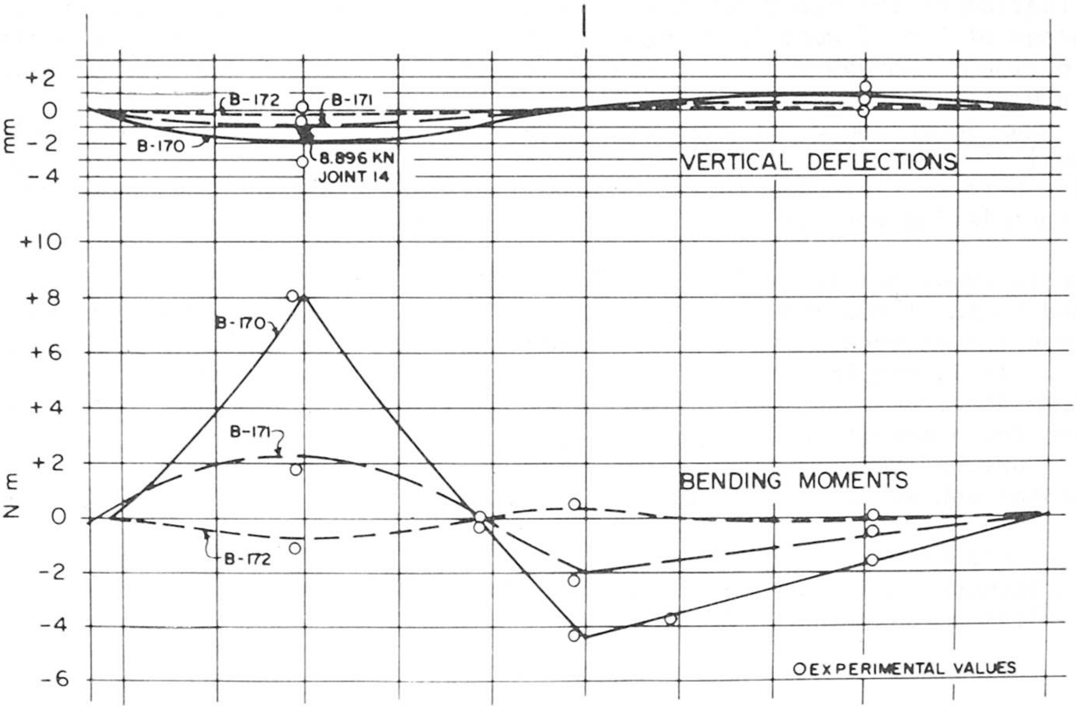


Fig. 10 Analytical and Experimental Moments and Deflections for Load at Joing 13
Small Scale Structure without Deck and with Deck



STRUCTURE WITHOUT DECK



STRUCTURE WITH DECK

Fig. 11 Analytical and Experimental Moments and Deflections for Load at Joint 14
Small Scale Structure without Deck and with Deck

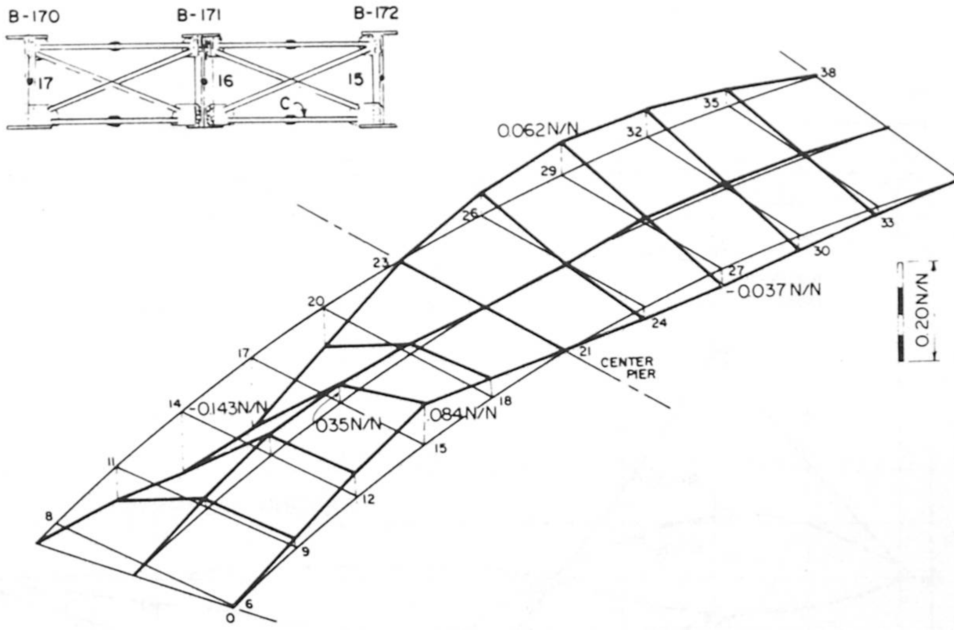


Fig. 12 Influence Surface for Force in Diaphragm Member C between Joints 15 and 16

These effects, particularly the second, introduce major problems in experimental verification of the diaphragm bar forces. Three interior diaphragms and the end diaphragm of Span 7 were instrumented with SR-4 strain gages. In the general case of low diaphragm stress it was extremely difficult to accurately measure strain differences. In many members the force was less than 200 Newtons. For upper and lower chord members this represents less than 10 microinches (250 microns) of strain, the lowest division on the strain indicators. When the strains were sufficiently large the analytical and experimental results showed good correlation for most members.

Figure 13 shows diaphragm force conditions for the diaphragm between Joints 12, 13, and 14 for three loading conditions. The conditions for steel framing loading were chosen because this loading represents distributed loading on each girder. It is not intended to represent forces in the diaphragm members resulting from the steel framing in the field because construction techniques can determine force conditions during erection. As shown, where the loading is of a uniform nature there is close correlation between theoretical analysis and experimental analysis.

For the condition of a concentrated load at a joint of the diaphragm, the correlation between theory and experiment appears very good for most members. The theoretical forces range from about 1.5 percent of the applied load to 21 percent.

7. CONCLUSIONS

The overall excellence of the correlation between experimental and analytical moments for many loading cases, both with and without a composite concrete deck, show that (1) a small scale geometric model can be constructed and can give meaningful results, it is an excellent tool for the study of bridges; and (2) the three dimensional analysis and computer program developed at Syracuse University can accurately predict the stress condition in horizontally curved (or straight) bridges both with and without a deck.

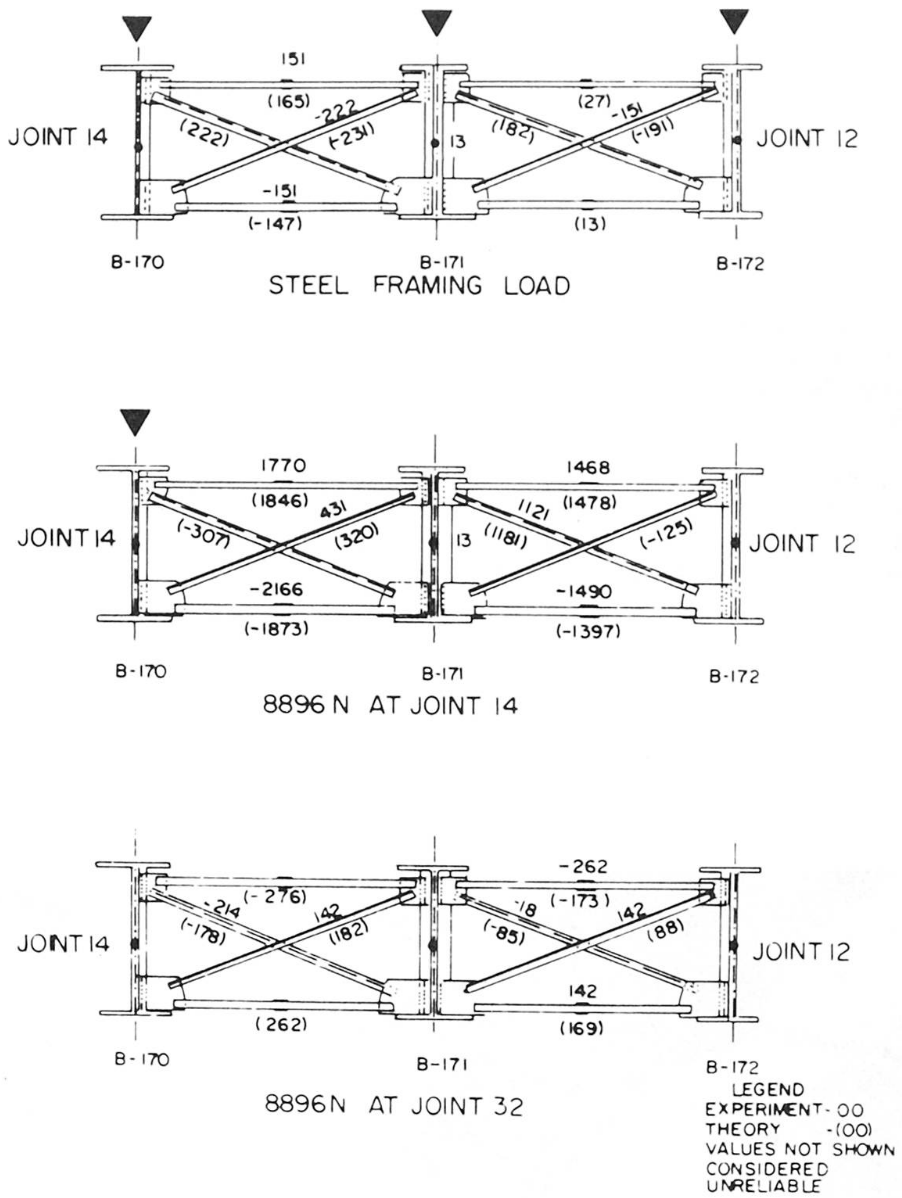


Fig. 13 Analytical and Experimental Forces in Diaphragm Members

8. ACKNOWLEDGEMENTS

The work described here is a portion of a cooperative research program involving a Consortium of University Research Teams (CURT). The studies were funded through 25 participating states and the Federal Highway Administration of the U.S. Department of Transportation. The cooperation of representatives of the participating states and the Federal Highway Administration is greatly appreciated.

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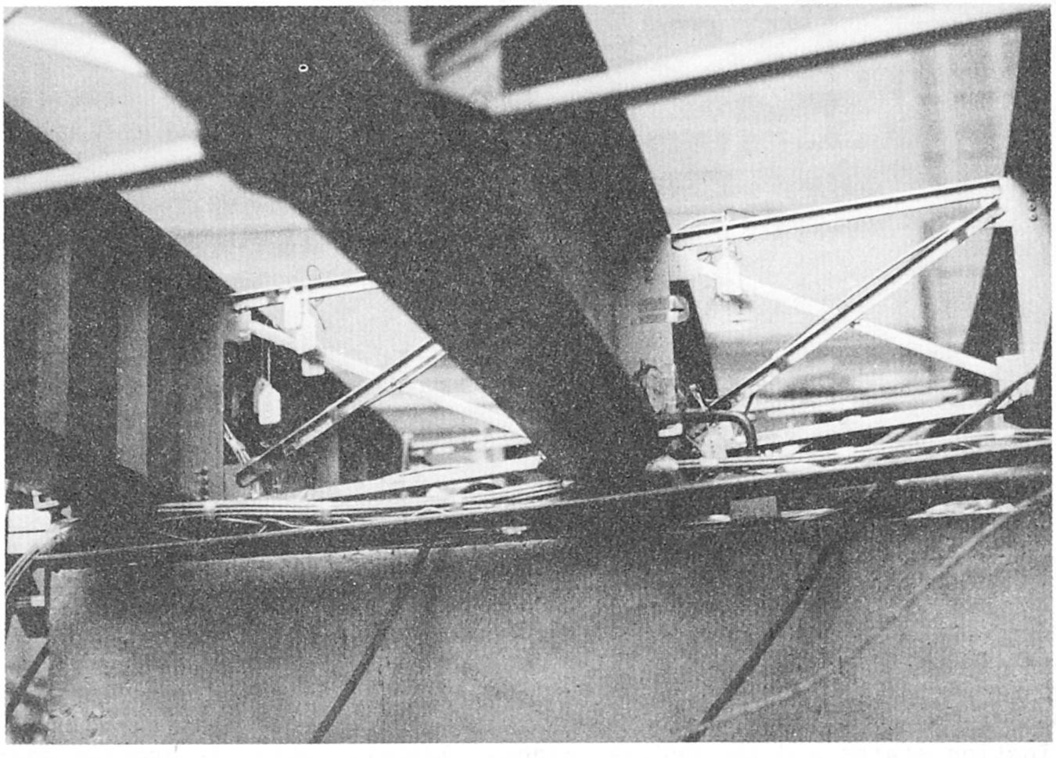

Fig. 14 Intermediate Diaphragm at Center Pier

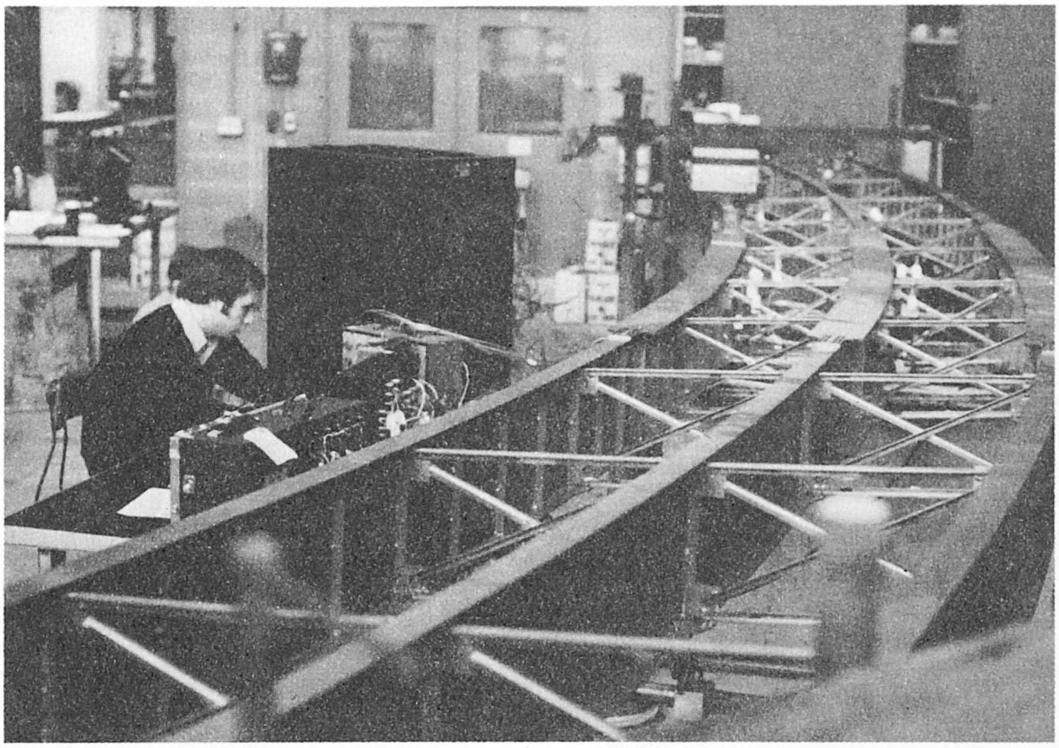

Fig. 15 Steel Framing of the Small Scale Seekonk River Bridge

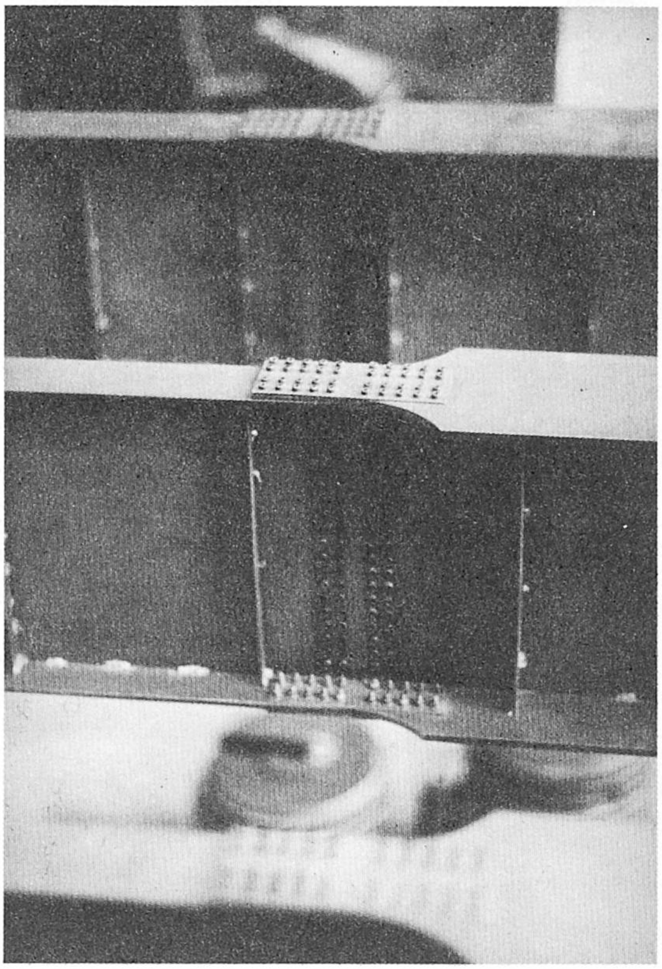

Fig. 16 Flange and Web Splices in the Small Scale Bridge

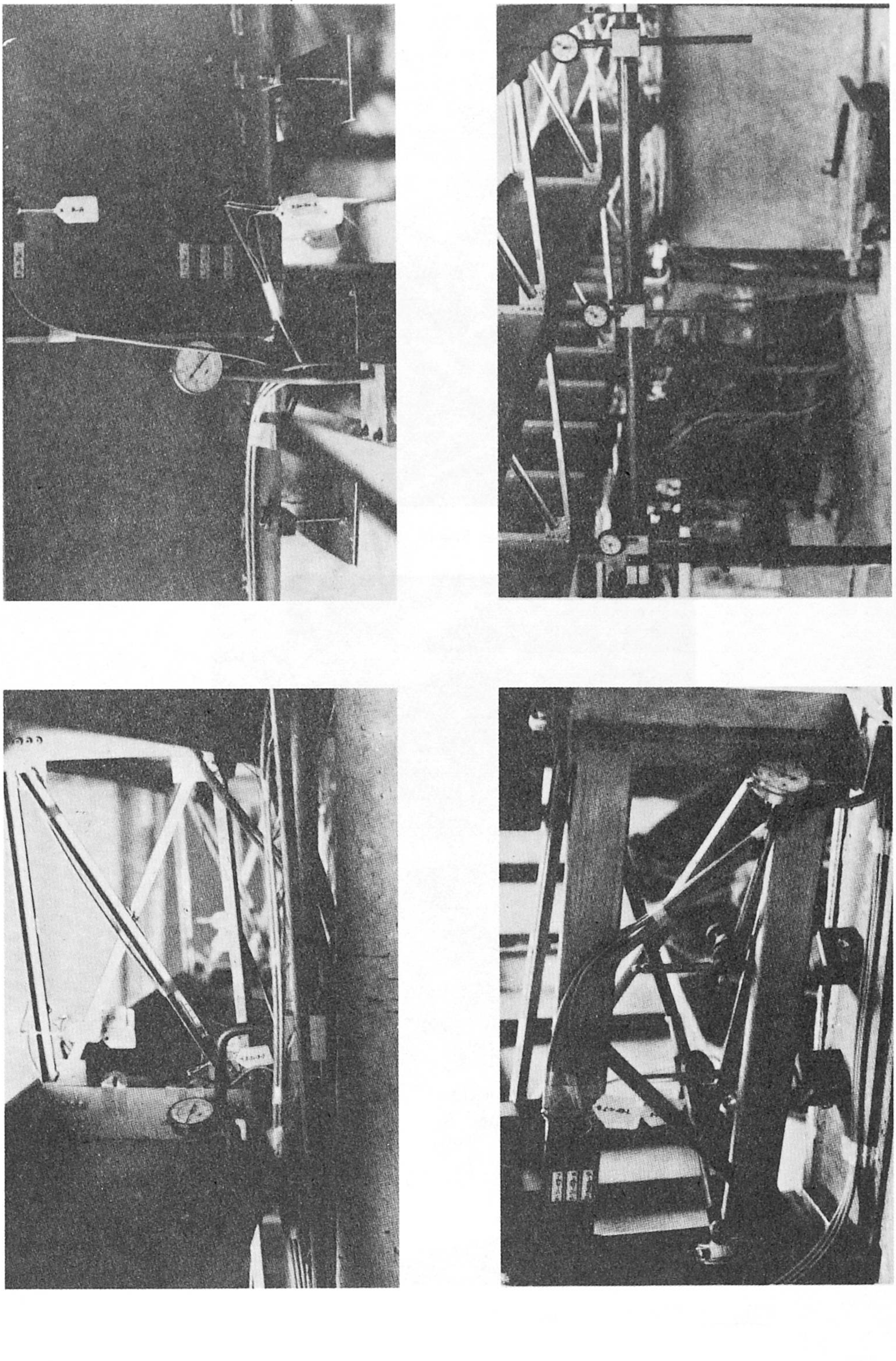

Fig. 17 Strain Gages and Dial Gages on the Small Scale Structure