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Response of orthotropic bridge decks to highway loadings

Harry Morris Rexin
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AN ABSTRACT OF THE THESIS OF Harry Morris Rexin for the Master of Science in Applied Science presented July 27, 1973.

Title: Response of Orthotropic Bridge Decks to Highway Loadings

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This thesis documents the fabrication, testing techniques, and response of a plastic scale model of an orthotropic bridge deck. To economically investigate a two-lane steel bridge deck, plastic Acrylic^R was used as a modeling material. Welds were simulated with dichloromethane, a capillary action solvent, and PS-30, a polymerizable cement. Deflections were measured with laboratory dial gauges while strains were monitored with strain gauges mounted on the deck.

The response of the deck to AASHO vehicle axle loads was compared with a discrete element computer program used to analyze orthotropic bridge decks continuous over flexible supports. Results indicate good correlation between measured and computed values for deflection and strains.

RESPONSE OF ORTHOTROPIC BRIDGE DECKS
TO HIGHWAY LOADINGS

by
HARRY MORRIS REXIN

A thesis submitted in partial fulfillment of the
requirements for the degree of

MASTER OF SCIENCE
in
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July 1973

TO THE OFFICE OF GRADUATE STUDIES AND RESEARCH:

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TO MY WIFE

JEAN MARIE REXIN

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Portland State University
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July 1973

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NOMENCLATURE

A_m	= Area of model
A_p	= Area of prototype
D_x	= Flexural rigidity of orthotropic plate per unit width in the x-direction
D_y	= Flexural rigidity of orthotropic plate per unit width in the y-direction
E_m	= Modulus of elasticity for model material
E_p	= Modulus of elasticity for prototype material
H	= Effective or reduced torsional rigidity of orthotropic plate per unit width
h_x	= Increment length in the x-direction in the discrete element model
h_y	= Increment length in the y-direction in the discrete element model
L_m	= Model lengths
L_p	= Prototype lengths
$P(x,y)$	= Magnitude of distributed load on plate at location (x,y)
P_p	= Load on prototype
P_m	= Load on model
S_ϵ	= Strain scale factor
S_p	= Load scale factor
w	= Vertical displacement of orthotropic plate
Δ_m	= Model deflections
Δ_p	= Prototype deflections

ϵ_m	= Strains in model
ϵ_p	= Strains in prototype
	= Length of small fiber on the tensile side of a beam section
σ_m	= Model stresses
σ_p	= Prototype stresses

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CHAPTER 1

INTRODUCTION

Definitions

Generally, an orthotropic bridge deck consists of a flat, thin steel plate, stiffened by a series of closely spaced longitudinal ribs at right angles to the floor beams (23)* (Fig. 1.1). The term orthotropic, the abbreviated combination of orthogonal-anisotropic, describes a continuum that has varying elastic properties in either two (plates) or three mutually perpendicular directions.

Orthotropy, if due to the physical structure of the material, is called natural orthotropy. An example of this would be wood which has different stiffnesses parallel and perpendicular to the grain due to the orientation of its cells. Technical orthotropy is a term used in structural engineering that describes: (a) structural elements composed of a single material which have different rigidities in two orthogonal directions owing to their geometry, and (b) structural elements composed of two or more materials. The structural elements dealt with in this investigation fall in category (a) and will be referred to simply as "orthotropic."

Historical Development

In the conventional bridge deck design the following members are assumed to act independently to carry the vertical loads:

*Numbers listed refer to references at the end of this thesis.

(a) A slab which transmits the loads to a series of longitudinal beams called stringers or ribs; (b) Stringers which are carried by floor beams; (c) Floor beams supported by the main girders; (d) Main girders which transmit their load to the bridge supports. The main disadvantage of this type of design lies in the fact that each element is thought to fulfill a separate function resulting in wasted material and an increase in the dead weight of the bridge. Actually, the above elements of the bridge system resist the load as an integral unit; when a load is placed at any point on the bridge, the decking and floor beams distribute it to the main girders in proportion to the relative rigidities of the different parts of the structure.

The first bridge that utilized the deck plate, stringers (ribs) and floor beams acting together, was an overpass at Jungingen, Germany, in 1934; true economy, however, was not achieved because the deck and main carrying members were analyzed as separate elements. The main advantage was its shallow depth which gave it a slender appearance.

During the same decade the American Institute of Steel Construction was experimenting with a similar type of deck construction known as the battled deck floor. It was from tests conducted at Lehigh University on scale models that a greater reserve strength than that predicted by bending theory was found in the deck plating (23). It was not until after World War II, when due to a shortage of steel and a need to replace many long span bridges in Germany, that the full economic realization of orthotropic design was recognized. The deck was

considered to fully participate in the stress of the main carrying members of the bridge.

The first bridge using this concept was the Kurpfalz Bridge over the Neckar River in Mannheim, Germany, which was opened to traffic in 1950 (19). Other important long-span bridge structures followed in quick succession. But it was not until 1960 that work was begun on the first orthotropic deck bridge in the Western hemisphere with the Port Mann Bridge over the Fraser River in Vancouver, B.C., Canada (13). In the United States, the Poplar Street Bridge over the Mississippi River in St. Louis, Missouri (22), built in 1966, spurred interest in orthotropic bridge construction, and other bridges of the same type followed. Major orthotropic bridges recently completed in the United States are the San Mateo-Hayward Bridge across South San Francisco Bay (8), the San Diego-Coronado Bridge (7), and the Fremont Bridge in Portland, Oregon (14), which is nearing completion.

Long before the advent of orthotropic deck bridges theoretical studies were conducted on orthotropic plates. The first mathematician to address the problem of anisotropic bodies was Cauchy (4), who in his paper published in 1828, gave generalized elasticity equations. The first application of the theory of anisotropy to a structural element, such as a plate, was attempted by Gehring (11) in 1860. Other theoretical investigations were carried out by Boussinesq (3) 1879, Voigt (25) 1910, and Geckeler (10) 1928. The first comprehensive

treatment of orthotropic plates was presented by Huber (16) in 1914.

It was from his works that the solution to orthotropic plate problem was developed. The following equation, known as "Huber's equation", gives the relationship between deflection and loading:

$$D_x \frac{\partial^4 w}{\partial x^4} + 2H \frac{\partial^4 w}{\partial x^2 \partial y^2} + D_y \frac{\partial^4 w}{\partial y^4} = P(x,y) \quad [1.1]$$

This is a non-homogenous differential equation because the deflection, w , does not appear in the function $P(x,y)$. D_x , D_y , and H are the rigidity coefficients while the function $P(x,y)$ is the loading intensity at any point. The solution involves first solving the corresponding homogenous differential equation where $P(x,y) = 0$ and superimposing this general solution with a particular solution of the non-homogenous equation. The result will vary depending on the relations between the three rigidities D_x , D_y , and H .

To treat a stiffened plate as an equivalent orthotropic plate, an assumption is generally made that the spacing of ribs is small relative to their length between supports. In 1946, Guyon (12) proposed a method of applying orthotropic plate theory to a bridge deck; this was followed by Massonnet (17) who took into account the effect of torsional rigidity. It was in conjunction with the design of the Cologne-Muelheim Bridge in 1951, that Cornelius (6) first applied Huber's equation to the problem of computing stresses. The most successful method of the application of Huber's equation was developed by

Pelikan and Esslinger (21). This method was adopted in the AISC Design Manual for Orthotropic Steel Plate Deck Bridges (2).

Although there appears to be an abundance of theoretical studies on the subject of orthotropic plates, experimental investigations known to date, most of which have been conducted in conjunction with the construction of major orthotropic bridges, are somewhat limited. Such tests have been carried out in different countries, primarily on prototypes. To overcome both physical and economic difficulties of testing prototypes, model tests on bridge structures have been successfully utilized by several investigators. Heins and Hails (15) used a curved stiffened plate model to show the validity of the mathematical model and the resulting computer program for a bridge with curved girders. Naruoka, Okabe, and Hori (20) made an experimental study of an orthotropic bridge model with torsionally soft ribs to check a proposed mathematical model. Troitsky and Azad (24) built and tested a plastic scale model of an orthotropic box girder with open ribs.

Purpose of Investigation

A two year-study of the behavior of torsionally stiff orthotropic decks was conducted recently by Erzurumlu and Toprac (9). The experimental program consisted of prototype tests of deck panels simply supported by unyielding floor beams. In contrast, this thesis evaluates the response of a multiple panel deck supported by flexible floor beams. In order to establish further confidence in the discrete

element model and computer program proposed by above investigators, to simulate highway loadings, and to keep experimental costs to a minimum, a scale model of plastic was considered to be the most appropriate.

The object of this investigation may be summarized as follows:

- (1) To develop adequate scale models of plastic for orthotropic steel bridge decks continuous over flexible floor beams.
- (2) To verify the adequacy of a discrete element computer program (18) used in Reference 9, by subjecting the bridge model to AASHO vehicle loadings.

CHAPTER 2

EXPERIMENTAL PROGRAM

The dimensions of the model to be investigated were chosen so that a two-lane steel orthotropic bridge deck would be simulated. The material sizes were selected on the basis of availability of plastic sheets. Although the primary objective of the investigation was not to simulate an existing deck, the dimensions as shown in Figs. 2.1 through 2.3, with the exception of the box girder, fell very near those of the Poplar Street Bridge (22) and the Fremont Bridge (14) when a scale factor of 4 was applied.

Material

The material used was Acrylite^R, an acrylic sheet manufactured by the American Cyanamid Company in Wayne, New Jersey. Its chemical name is poly(methylmethacrylate). The modulus of elasticity was determined by standard coupons taken from the plastic sheets used in fabricating the deck. Its average value was found to be 394 ksi. The Poisson's ratio was taken as 0.35 as suggested by the manufacturer.

Equations of Similitude

Principles of similitude governing structural models are well documented (26). Structural models may be classified as direct and indirect. Direct models are made of the same material as the prototype and simulate the true response of the prototype including ultimate strength. On the other hand, indirect models simply represent the

response of the prototype within the elastic range, and need not be fabricated of the same material as the prototype.

To illustrate the basic relationship governing the similitude of geometry and loading, the following equations are provided for the sake of clarity. By using a scale factor of 4, all lengths in the deck model are one-fourth the lengths of the prototype:

$$L_m = \frac{L_p}{4} \quad [2.1]$$

where L_m = length in the model;

L_p = length in the prototype.

Using the same reasoning, model deflections are one-fourth the prototype deflections:

$$\Delta_m = \frac{\Delta_p}{4} \quad [2.2].$$

where Δ_m and Δ_p are deflections in the model and prototype respectively.

Strains are related by the strain scale factor:

$$S_\epsilon = \epsilon_p / \epsilon_m$$

where S_ϵ = strain scale factor;

ϵ_p = strain in prototype;

ϵ_m = strain in model.

Considering the deformation Δ of a small fiber of some length l on the tensile side of a beam section:

$$\Delta_p = \epsilon_p l_p \quad [2.3]$$

$$\Delta_m = \epsilon_m l_m \quad [2.4]$$

By substituting Eqs. [2.1] and [2.2] into [2.4]:

$$\frac{\Delta_p}{4} = \epsilon_m \frac{L_p}{4} \quad [2.5]$$

The substitution of Eq. [2.3] into Eq. [2.5] gives:

$$\epsilon_p = \epsilon_m \quad [2.6]$$

Therefore, the strain scale factor equals one in a true model where strains at corresponding points in model and prototype are equal, provided the loads are scaled properly.

Since $S_\epsilon = \frac{\epsilon_p}{\epsilon_m} = 1$, loads can be scaled using the following relationships:

$$P_p = \sigma_p A_p$$

$$\frac{\sigma_p}{\sigma_m} = \frac{E_p}{E_m} \quad \text{and}$$

$$A_m = \frac{A_p}{(4)^2}$$

where A_m and A_p = areas of the model and prototype respectively;

E_m and E_p = the modulus of elasticity of model and prototype respectively;

σ_m and σ_p = the stress in the model and prototype respectively;

P_p = load on the prototype.

From which:

$$P_p = \left[(A_m)^4 \right] \left[\sigma_m \frac{E_p}{E_m} \right] \quad [2.7]$$

$$\text{Since } P_m = \sigma_m A_m$$

$$S_p = \frac{P_p}{P_m} = (4)^2 \frac{E_p}{E_m} \quad [2.8]$$

where S_p = load scale factor;

P_m = load on the model.

If the modulus of elasticity of steel is 29,000 ksi and that of plastic is 394 ksi, then:

$$P_m = \frac{P_p}{S_p} = (0.000849) P_p \quad [2.9]$$

This means that a load of 100 lbs used in the model test is equivalent to a 117.8 k load on the prototype. The total weight of one HS20 truck is 72 k (Fig. 2.4) (1).

Loading

Scale models of actual trucks would require a minimum length of 7 ft on the deck. Because the length of model deck was only 12 ft, it was considered sufficient to simulate only single wheel and axle loads. The size of the loading pad was taken as 6 in. by 2.5 in., which is equivalent to an AASHO HS20 truck wheel contact area (Figs. 2.5, 2.6, and 2.7). This area is determined assuming that the tire is in direct

contact with the deck plate, without considering pavement, thus representing the worst condition for local stresses in the vicinity of the loaded area. The applied load was transmitted to the deck through a 1/2-in. neoprene pad and a steel plate as shown in Fig. 2.8. A summary of all loading positions is shown in Fig. 2.9.

Strain and Deflection Gauges

The strain gauges used were Micro-Measurements, Inc. EA-06-250BG-120, which are general purpose gauges intended for static and dynamic stress analysis. To overcome the heating problem and the resulting errors caused by the poor heat sink quality of plastic, a method of pulsing, which will be discussed subsequently under testing procedure, was used.

The positions of the strain and deflection gauges are shown in Figs. 2.10 and 2.11 respectively. The gauge installation procedure as suggested by the manufacturer was followed closely. After installation, a piece of cellophane tape was placed over each gauge to protect the lead wires. All lead wires were cut at equal lengths (12 ft) to ensure that variable lead resistance would not be a factor during testing. The strain gauges were monitored by a Datran II strain recording system which consists of a Model 1330 10-channel scanner, a Model 321 strain indicator, and a Franklin 1200 Hi-Speed Printer (Fig. 2.12). A gauge factor of 2.10 was used for all strain gauges.

Deflections of the deck were measured with dial gauges to an accuracy of 0.001 inch. The gauges were mounted on a bridge supported

on two sides by metal stands (Fig. 2.13). Support deflections also were monitored and used later to obtain corrected span deflections.

Testing Procedure

In testing models made of plastic two problems are encountered which are not significant in metal models: (a) As mentioned earlier, the temperature of the strain gauges increases due to the insulating characteristics of the plastic; (b) A significant amount of creep occurs.

To avoid heating, strain gauges were only activated during an actual reading. In this manner the presence of current in each circuit was limited to just a fraction of a second. To efficiently take the readings, the Franklin 1200 Hi-Speed Digital Printer was connected to the strain indicator, thereby enabling all ten channels to be read and recorded within one second.

In tests conducted on independent specimens it was found that significant creep occurred immediately after the application of a load. These tests also showed that after approximately five minutes 95 percent of the creep had taken place; therefore, it was decided to take all readings after a five-minute pause, a procedure followed in determining the value of the modulus of elasticity. The testing procedure was as follows:

1. Take initial strain and deflection gauge readings.
 2. Start timer.
 3. Apply weights.
 4. Take immediate readings.
-

5. Wait five minutes.
6. Take readings.
7. Apply next loading increment and repeat cycle until
five increments at 20 pounds each have been added.

By using 20-pound increments a total of one hour was necessary to complete one test as the deck was unloaded the same way. Figure 2.12 shows the complete set-up during one of the tests. One person took all dial gauge readings which were called out to a recorder to reduce the chance of error.

CHAPTER 3

FABRICATION

The orthotropic deck model was built of standard 1/16-in. and 1/8-in. thick sheets of acrylic. A laboratory check of these thicknesses revealed that the 1/16-in. sheets were 0.061 in. thick. This value was used in determining the section properties of the deck model. All connections were made by a gluing process, thus simulating welds in an actual steel deck. Prior to fabrication of the model, test beams were built using proposed gluing techniques. These beams were subjected to heavy loadings which verified the adequacy of the welds. Due to the unavailability of desired lengths of material, splices were required.

Splices

The deck plate required a 12-ft x 6-ft sheet of 1/8-in. acrylic, but the largest size available locally was 10 ft x 6 ft. Therefore, a splice two feet from the end of the deck was considered. Because the 10-ft x 6-ft sheet was 6 in. oversized, the splice actually occurred 18 in. from the end midway between the last two floor beams. This was considered acceptable as the splice was 3 ft from the points being monitored and in an area where the stresses were low.

In order to provide continuous ribs, a 12-ft length of 1/16-in. material was needed. Again, the maximum length that could be purchased locally was 6 ft. Under the circumstances, it was determined that a center splice in the ribs would be acceptable as the deck plate is

continuous over the rib splices. The tie girder splices were made 18 in. from either end, and staggered so that no two splices would coincide.

All splices were made butt-joint style. When a good fit was obtained, dichloromethane was used as a bonding agent; where there were inconsistencies in the fit, PS-30 was used.

Gluing Techniques

Dichloromethane, a capillary action solvent that produces a medium strength joint, can be used only when parts are closely fitted with no visible gaps. The solvent is applied with a hypodermic needle along the edge of the joint. Capillary action draws the solvent between the parts, softening and swelling the plastic which then permits actual cohesion of the parts. After approximately five minutes, the solvent evaporates leaving a hard, clear joint. It was found that a thickened solvent could be produced by dissolving 3 percent clean plastic chips in the dichloromethane. This more viscous solvent had bonding characteristics similar to the unthickened solvent, but was more manageable when applying.

PS-30, a highly viscous polymerizable cement, has little or no solvent action, and produces good joint strength after four hours. PS-30 comes in two components, A and B. Component A is a viscous solution consisting of a monomer and a polymer, while component B is the catalyst. The shelf life of both components is six months; the pot life is only 25 minutes. Before mixing, the components should be

allowed to warm to room temperature for 12 to 24 hours. Proportions required to make PS 30 are:

Component A	95 grams or 95 cc
Component B	5 grams or 5 cc

After stirring for two minutes, the mixture was applied to the joint with a 13 gauge needle and 30 cc syringe.

Although it is recommended that a V-type joint be used in conjunction with PS-30 for maximum strength because of the small thickness of the parts, no V-grooved joints were used during fabrication. PS-30 was used only to overcome natural nonconformities in fit.

Cutting

The plastic sheets were cut to size by a 10-in. radial arm saw or a 12-in. table saw with a No. 88 plywood blade. To avoid handling the deck plate, it was trimmed to size in place using a 7-in. Skillsaw with a plywood blade.

The 1-1/2-in. strips used for the rib bottoms were cut on the radial arm saw (Fig. 3.1). Out of 24 strips cut, four were determined to be unsuitable for use and had to be cut again due to a variance in width. For cutting the 3-in. rib webs, the blade was set at a 14 degree angle providing the beveled edge needed for a proper fit to the rib bottoms and the deck plate (Fig. 2.2). Of the first six strips cut this way, three had to be discarded as they were as much as 1/4 in. out of alignment. At this point it was decided to use the 12-in. table saw to ensure a better edge alignment.

A total of 44 rib sides and 22 rib bottoms were cut. To assure that the ribs would be 12 ft in length when glued together, two sides and one bottom were bundled, taped, and then measured and trimmed off in 6-ft lengths with the radial arm saw.

Ribs

In order to hold the rib sides at a 14 degree angle while gluing to the bottoms, an eight-foot gluing jig was constructed to adequately accommodate the 6-ft rib lengths (Fig. 3.2). Foam rubber, 1/2 in. thick by 1-1/2 in. wide, was laid on the bottom of the jig to provide a spring-like effect so that the rib bottom plate would conform to the rib webs during the gluing process. The following step-by-step process was used in the fabrication of trapezoidal ribs:

1. Lay 1-1/2-in. plastic strip on foam rubber pad.
2. Place a 1-in. by 1/2-in. wood strip on top of the plastic strip.
3. Compress foam pad and clamp in place using large "C" clamps (Fig. 3.2).
4. Place rib webs along the two edges of the wooden jig on the rib bottom plate.
5. Using small "C" clamps fasten rib sides to sides of gluing form (Fig. 3.2).
6. Release large "C" clamps allowing foam rubber pad to gently push the rib bottom against the rib webs making a firm contact.

7. Apply dichloromethane to inside edge of rib using a hypodermic needle. (Fig. 3.3)
8. Wait 30 minutes; then release small "C" clamps and remove finished rib from jig.
9. Inspect rib for any opening along line, and, if necessary, reseal with PS-30.

Floor Beams

Fabrication of the floor beams consisted of four steps:

1. Cutting 8-in. by 6-ft strips for web plates.
2. Cutting openings in the web plates as shown in Fig. 3.4 to accommodate the continuous ribs.
3. Cutting five 1/8-in. x 2-1/2-in. by 6-ft flange plates.
4. Gluing flanges to the web.

Some problems were encountered in making the web openings for the floor beams. The sides of the 3-in. deep openings were cut with a radial arm saw, while the 1-1/2 in. bottom was cut with a coping saw. The same technique as in gluing the ribs was used to glue the web to the flange. The flange was placed on a foam rubber pad, and a series of clamps were used to form a jig to keep the web upright (Fig. 3.5). Small wedges were then slipped under the foam pad where needed to provide the proper fit. Gluing was accomplished by applying dichloromethane to both sides of the web. No PS-30 was needed.

Assembly of the Orthotropic Deck Model

After the fabrication of all the basic elements of the orthotropic

deck model was completed, it was extremely important to follow a well-planned sequence of assembly steps. The protective paper was first removed from the deck plate and the ribs were placed on the deck in their proper positions. To provide a means of pushing the deck plate up against the ribs a large foam rubber pad was placed under the deck (Fig. 3.6). The ribs were positioned by using the notched floor beams as spacers. When all ribs were in their proper position, weights were applied at various locations to provide a tight fit. Again, wedges were slipped under the pad to help make the deck conform to the ribs. All ribs were checked visually and with a thin feeler gauge for proper connection to the deck. The ribs were kept tight against the floor beam notches by means of thin wooden wedges which were manipulated by a 7-ft pole with a hook on the end.

Pursuant a lengthy process of assuring that every piece was properly positioned and fitted, ribs were glued to the deck using dichloromethane. This assembly was left undisturbed for 24 hours after gluing before removing weights and floor beams. Checks were made for places that were not welded properly, and such points were reglued using PS-30. After the rib plates were spliced at mid span, the floor beams were repositioned over the ribs. More filing of floor beam webs was needed in order to obtain a proper fit. In the event of some gaps ($1/16 \pm$ in.) which were too large to bond with PS-30, small plastic wedges were inserted prior to gluing. In order to keep the highly viscous PS-30 from pushing the wedges out during application, dichloromethane was used to spot-weld the wedges in place before applying PS-30.

The floor beams were held in place during the welding process by heavy steel bars lying over the tops of the ribs and clamped to supports at each end (Fig. 3.7). Weights were placed on top of the beam flange to help hold it against the ribs. The floor beam was then welded to the ribs and deck plate. After all floor beams were in place the deck was turned over to check the reliability of the welding (Fig. 3.8).

When it was determined that the welding of basic deck elements was adequate, the model was again turned over and the tie girders were added. The inside web of the tie girder was welded to the deck and floor beams first. The outside web was then installed, followed by centering the flange and welding with dichloromethane. The total weight of the finished assembly was approximately 125 pounds.

Supports and Loading Beam

The model deck was simply supported on four corners by 2-in. by 3-in. steel plates (Fig. 2.1). Two of these plates were placed on fixed rollers (simulating a pinned joint) while the opposite pair was placed on standard rollers.

The loading beam consisted of a 10-ft aluminum H-shape supported at loading point or points. Two weight hangers were provided on each end of the loading beam. Each calibrated weight consisted of a five-pound chunk of caulking lead. During testing, a number of weights were hung from the loading beam to obtain the desired load (Fig. 3.9).

CHAPTER 4

TEST RESULTS

The Computer Model

A discrete element model capable of handling orthotropic plate problems (18) was used for the analytical part of this investigation. The model (Fig. 4.1) represents the stiffness, geometry, and support conditions of an orthotropic plate continuous over flexible floor beams. The principal features of this method include the representation of structural members by a physical model of bars and springs which for analysis are grouped into two orthogonal systems of beams, and replacing the differential equations for orthotropic plates by their finite-difference equivalent. The problem then is reduced to solving a large number of simultaneous equations rather than one complex differential equation. The computer program developed from the above model permits the stiffness properties of the orthotropic deck and floor beams to be included on an individual basis, thus resulting in a more realistic representation of the bridge deck as an equivalent orthotropic plate. The bending stiffness of the plate in the x- and y-directions and the Poisson's ratio effects are represented by elastic blocks at the nodal points (Fig. 4.2). Support conditions are simulated by elastic springs of appropriate stiffness at each joint, while twisting stiffness is modeled by torsion bars connected to the rigid bars between elastic nodal blocks.

Member stiffness and load values are entered in the computer program by first dividing the slab into increment widths of h_x and h_y in the x- and y-directions respectively. Figure 4.3 shows the coordinates of the deck model under investigation. A joint is defined as the intersection of the station lines in each x- and y-direction, while a mesh is understood to be the area surrounded by four joints. Orthogonal flexural stiffnesses, D_x and D_y , are entered on a per unit width basis. The torsional stiffness, H , is input on a unit width basis for each mesh; its value is determined with the aid of a separate computer program following the method in reference (2). A listing of this program is provided in Appendix I(a). Applied loads are assumed concentrated at each joint; therefore, a distributed load must be proportioned among the surrounding joints on a tributary area basis. Unyielding supports are simulated by introducing very stiff springs (500 k/in.) at the joints on the support station line. Discrete members, such as floor beams and girders, can be entered on an individual basis by giving their total stiffness values. A typical computer output is given in Appendix I(b).

Idealized Sections

The only function of the tie girders was to provide a flexible support for the floor beams. Therefore, no attempt was made to properly scale the tie girders, as in an actual bridge their span lengths and cross-sectional dimensions would be much greater. Consequently, theoretical investigations by Chwalla (5) on determining the effective

width of the deck acting as the top flange of the tie girder could not be used. Instead this effective width was determined experimentally by measuring girder deflections for several symmetrical loading conditions and computing the required inertia needed to obtain this deflection. The tie girder inertia value was found to be 44.34 in.⁴ which corresponds to an effective deck width of 20 inches.

The effective widths of the orthotropic deck plate acting with a rib and floor beam were computed using the method outlined in the AISC Design Manual (2) and others (23). It was determined that, because of its narrow width, the tie girder would offer little or no rotational restraint for the floor beams; this justified using the full length of the floor beam as its effective span when computing the effective width. All idealized sections along with corresponding inertia and stiffness values are shown in Appendix II.

Calculated vs. Observed Deflections and Strains

(a) Deflections: All deflections were corrected to account for support movements during loading. This was done by using a linear interpolation of the settlements between supports. Although a biaxial stress field exists in the rib plates, the use of uniaxial strain gauges to determine strains at rib bottoms was justified through experimental observations made by Erzurumlu and Toprac (9). Linearity checks were made on deflections and strains for all loading conditions prior to plotting any profiles.

Computed and measured center floor beam deflections are shown in Figs. 4.4 through 4.9 for all loading conditions. At this juncture,

it should be noted that the mathematical model does not account for shear deformations. The deflection due to shear was computed for a 100-lb load applied at the center of the floor beam and was found to be 0.023 inches. If this correction is applied to the computed deflections of Fig. 4.4, the correlation between computed and measured deflections would be very good with the difference reduced to less than one percent. Examination of floor beam deflection curves for other loading conditions indicates that when the load is not directly over the floor beam there is closer agreement between computed and measured values. This substantiates the need for a shear deflection correction on floor beams, as less shear correction would be needed for smaller floor beam loads. It is interesting to note that good agreement exists between computed and measured deflections at the tie girder as long as the load is symmetrically located; however, as Figs. 4.8 and 4.9 indicate, a difference of approximately five percent exists when loads are positioned in the driving lanes. Thus the tie girder inertia value and corresponding effective deck width are dependent, among other things, upon the position of the load, which is what Chwallas' (5) theoretical investigations indicated.

Good agreement exists between experimental and computed deflections of the ribs in Section 1-1 (see Fig. 2.11) for the loading conditions shown in Figs. 4.10 through 4.17. Generally measured deflections are slightly more than those computed with the difference seldom exceeding five percent. Again, shear deflections may account for

some of this discrepancy as the difference is usually greater in the center of the deck or directly under the loading pads. Another possible cause of discrepancy is the localized distortion effect in a rib produced by the moving together of the deck plate and the bottom rib flange resulting in a lowered moment of inertia and section modulus when a rib is under load. In Fig. 4.11 the computed deflection directly under the load is greater than the measured deflection. This behavior is believed to be caused by the fact that the wheel load is input as a point load due to the limitation of the mesh size. Because the 6-in. long loading pad is centered over a node, it cannot theoretically affect adjoining nodes. In actual behavior, however, the load is distributed to some extent to the adjoining nodes, thus accounting for the difference between measured and computed deflections on the adjoining ribs. An examination of Fig. 4.17 reveals the same type of loading condition resulting in the same relationship between computed and measured deflections. The extent of the discrepancy is not great in this case as the load on each node is only one-half that of Fig. 4.11. Generally when the load is over the floor beam the rib action is fairly uniform (Figs. 4.10, 4.13, 4.15, and 4.16). When the load is in an adjoining panel, centered between floor beams (Fig. 4.12), there is a slight rise in the center rib. The decreased magnitude of the computed deflection again is due to the load being input as a point load. A check of the strains at this point, both theoretical and measured, reveals that the bottom fiber of the loaded rib is in compression. In contrast, the adjoining rib bottoms are in tension,

and thus verify the shape of the curve. The deflections and strains for computer coordinate (6,6) (floor beam bottom) are given in Table I. Deflections again vary depending on the position of the load with the greater percentage difference being when the load is near the floor beam.

(b) Strains Overall strains vary anywhere from 1 to 49 percent with computed values generally higher than measured values. The large discrepancy between computed and measured strains is probably due to the following causes:

1. Instrumentation error.
2. Warping of the deck due to uneven support settlements.
3. Idealization of the structure as a discrete element model.
4. In the case of ribs, the assumption of a uniaxial stress field in the rib bottoms.

The instrumentation error could be as high as $\pm 20 \mu\text{in./in.}$ and would account for a larger percent difference between computed and measured values in low ranges. Therefore, readings less than $100 \mu\text{in./in.}$ would be too small to verify the adequacy of the discrete element computer program.

The warping effect could also cause plus or minus strain errors, depending on the magnitude and location of the support settlements.

A safe assumption is that the more warping, the greater discrepancy between computed and measured strains. As an illustration, referring to Table I, the 30 percent difference in loading condition F could be due to warping because of the unsymmetrical loading in that case. An examination of the support deflections reveals that Gauge No. 7 drops 0.049 in. while the diagonal corner Gauge No. 9 drops only 0.011 inches. This indicates a warping effect and a possible explanation of the 30 percent discrepancy.

While the accuracy of the discrete element model could be improved by using a finer grid, the mesh size chosen was considered adequate for the purpose of this investigation and prevented excessive use of computer time. Because the theoretical data generated for deflections and bending moments are average values, there will likely be some differences between experimental and computed values. The errors introduced by assuming a uniaxial stress field in the rib bottoms would have some influence on the computed versus measured relationship, which would be more apparent when the ribs are heavily loaded.

Computed and measured strains for most loading conditions are given in Tables II through IX. Keeping in mind the previously mentioned possible sources of error, there is generally a good agreement between theory and test results. It is interesting to note that when a load is directly over a rib (Tables III and VIII), the computed strain is considerably greater than measured strain--38 to 49 percent.

This can be partly attributed to the load being input as a concentrated load, and partly to the excessive distortions of the rib plate directly under the load. This localized distortion effect was also noted in the investigation reported in Reference (9). A check of the floor beam strains near the girder (Table II) Gauge I-7, indicates there is tension present at that location. Although the strains are too small to be conclusive, it indicates the assumption made earlier, which stated that the tie girder offered little or no rotation restraint for the floor beam, was reasonable.

It could be concluded that, within the limits of the experimental program, the orthotropic deck responds as predicted by a discrete element computer program in the elastic range.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

In view of the findings documented in Chapter 4 it may be concluded that:

1. Plastic sheets of acrylic can be used as a modeling material for the fabrication of orthotropic deck models. Ordinary woodworking tools are effectively used in the deck construction, thus keeping costs to a minimum.
 2. Dichloromethane and PS-30 are compatible and effectively simulate welds in steel orthotropic decks when tests are conducted within the loading range of this investigation.
 3. Creep effects of plastic do not adversely influence test results when proper testing procedures are used.
 4. Indirect models can be constructed from plastic acrylic material to test structures as complex as orthotropic decks.
 5. The discrete-element model proposed by Matlock and Panak (18) is effective in predicting the moments and deflections along grid lines of multiple panels of orthotropic decks on flexible supports.
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Recommendations

Future investigations involving orthotropic bridge decks should, based on this study, be concerned with the following:

1. A computer program using a finer mesh size should be run on this model to see if the results would change significantly.
2. The deck should be supported on concrete blocks so that warping would not be a factor in recorded deflections and strains.
3. The box girders should be properly scaled so that a check on its torsion restraint could be made.
4. The model should be extended to a length such that a full AASHO Highway vehicle could be simulated as a live load.
5. Investigations should be conducted on the interaction of the rib and floor beam by extensively gauging the areas of intersection.
6. Decks using different types of ribs (open, biserrated, etc.) should be modeled and their response compared with those obtained from closed rib models.
7. Orthotropic decks on flexible supports should be modeled with steel as a means of checking the ultimate strength of the system.

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FIG.1.1 ORTHOTROPIC STEEL PLATE DECK WITH CLOSED RIBS

FIG. 2.1 DECK DIMENSIONS

LONGITUDINAL RIB

FLOOR BEAM

FIG. 2.2 LONGITUDINAL RIB AND FLOOR BEAM

FIG. 2.3 BOX GIRDER

FIG. 2.4 LOADING PAD SIZE FOR HS20

CONTACT AREA OF SOLID TIRE
SUBSTITUTED EQUIVALENT AREA

FIG. 2.5. CONTACT AREA
OF 11 1/4 TON WHEEL

A: ACTUAL CONTACT
AREA OF ONE TIRE AT
THE TOP OF THE WEAR-
ING SURFACE. ASSUMED
CONTACT AREA AT THE
TOP OF THE WEARING
SURFACE, $2c \times 2g$.

ASSUMED LOADED AREA
AT THE TOP OF THE
STEEL DECK PLATE,
 $2c \times 2g$.

WHEEL LOAD IN KIPS	INCHES	INCHES
8	20	10
12	20	10
16	24	10

FIG. 2.7 TABLE

FIG. 2.6 WHEEL LOAD
DIMENSIONS

Fig. 2.8 Loading Pads and Beam

A - PAD AT ϵ RIB & ϵ OF FLOOR BEAM

B - PADS AT ϵ RIB & BETWEEN FLOOR BEAMS

C - PADS BETWEEN RIBS & AT ϵ FLOOR BEAM

D - PADS BETWEEN RIBS & BETWEEN FLOOR BEAMS

E - PADS AT ϵ RIBS & ϵ OF FLOOR BEAM

F - PADS AT ϵ OF RIBS & BETWEEN FLOOR BEAMS

FIG. 2.9 LOADING CONDITIONS & PAD LOCATIONS

= GAUGE ON RIB
 I GAUGE ON FLOOR BEAMS
 FIG. 2.10 LOCATION OF STRAIN GAUGES

FIG. 2.11 LOCATION OF DIAL GAUGES

Fig. 2.12 Test Set-Up During Testing

Fig. 2.13 Bridge Used in Deflection Measurements

Fig. 3.1 Radial Arm Saw Used for Cutting Plastic Sheets

Fig. 3.2 Gluing Jig and Clamps

Fig. 3.3 Application of Dichloromethane to Rib Seams

Fig. 3.4 Cutting Openings in Web Plates

Fig. 3.5 Jig Used in Gluing Floor Beam Web to Flange

Fig. 3.6 Deck Assembly Prior to Gluing

Fig. 3.7 Assembly Used to Hold Floor Beam In Place Prior to Gluing

Fig. 3.8 Turning the Deck Over

Fig. 3.9 Applying Loading Increments During Testing

FIG. 4.1 DISCREET ELEMENT MODEL OF ORTHOTROPIC PLATE (18)

FIG. 4.2 TYPICAL JOINT FROM DISCREET ELEMENT MODEL (18)

FIG. 4.3 COMPUTER GRID

FIG. 4.4, CENTER FLOOR BEAM DEFLECTION, LOAD COND. A

FIG. 4.5 CENTER FLOOR BEAM DEFLECTION, LOAD COND. B

FIG. 4.6 CENTER FLOOR BEAM DEFLECTION, LOAD COND. C

FIG. 4.7 CENTER FLOOR BEAM DEFLECTION, LOAD COND. D

FIG. 4.8 FLOOR BEAM DEFLECTION, LOAD. COND. E

FIG. 4.9 FLOOR BEAM DEFLECTION, LOAD. COND. F

FIG. 4.12 SECTION 1-1, DEFLECTION RIB PROFILE, LOAD. COND. B,

FIG. 4.13 SECTION 1-1, DEFLECTION RIB PROFILE, LOAD. COND. C

FIG. 4.15 SECTION 1-1, DEFLECTION RIB PROFILE, LOAD COND. D

FIG. 4.16 SECTION 1-1, DEFLECTION RIB PROFILE, LOAD COND. E

FIG. 4.17 SECTION 1-1, DEFLECTION RIB PROFILE, LOAD COND. F

TABLE I

FLOOR BEAM DEFLECTIONS FOR GAUGE LOCATION I-5
FLOOR BEAM STRAIN READINGS FOR GAUGE LOCATION I-9

LOADING CONDITION	A						B						C						D						E						F							
	Comp.						Meas.						Comp.						Meas.						Comp.						Meas.							
Deflection (I-5) Inches	0.114	0.124	0.135	0.151	0.113	0.124	0.132	0.145	0.110	0.116	0.130	0.142																										
Computed Measured	0.91		0.89		0.91		0.91		0.95		0.92																											
Strain (I-9) Microinches	175	206	618	531	157	173	446	380	122	137	338	260																										
Computed Measured	0.85		1.16		0.87		1.17		0.89		1.30																											

P = 100 LBS.

TABLE II
STRAIN READINGS FOR LOADING CONDITION A

	GAUGE NUMBER	Computed μ in./in.	Measured μ in./in.	$\frac{\text{Computed}}{\text{Measured}}$
Center Floor Beams	I-1	873	789	1.11
	I-2	679	660	1.03
	I-3	510	495	1.03
	I-4	361	329	1.10
	I-5	229	202	1.13
	I-6	*	*	-
	I-7	*	*	-
Section 1-1 (Ribs)	II-3	205	208	0.99
	II-6	202	229	0.88
	II-8	194	199	0.97
	II-9	183	173	1.06
	II-10	170	147	1.16

* Too small and within the range of tolerance of recording instruments.

P = 100 LBS

TABLE III
STRAIN READINGS LOADING CONDITION B

	GAUGE NUMBER	Computed μ in./in.	Measured μ in./in.	Computed Measured
Center Floor Beams	I-1	599	599	1.07
	I-2	486	495	0.98
	I-3	375	379	0.99
	I-4	272	251	1.08
	I-5	176	156	1.13
	I-6	*	*	-
	I-7	*	*	-
Section 1-1 (Ribs)	II-3	2429	1635	1.48**
	II-6	453	593	0.76
	II-8	238	242	0.98
	II-9	202	174	1.16
	II-10	*	*	-

* Too small and within the range of tolerance of recording instruments.

** Wheel directly over rib.

P = 100 LBS

TABLE IV
STRAIN READINGS FOR LOADING CONDITION C

	GAUGE NUMBER	Computed μ in./in.	Measured μ in./in.	Computed Measured
Center Floor Beam	I-1	587	500	1.17
	I-2	601	559	1.08
	I-3	531	507	1.05
	I-4	377	326	1.16
	I-5	241	190	1.27
	I-6	*	*	-
	I-7	*	*	-
Section 1-1 (Ribs)	II-3	198	160	1.22
	II-6	196	184	1.07
	II-8	190	154	1.23
	II-9	180	133	1.35
	II-10	*	*	-

* Too small and within the range of tolerance of recording instruments.

P = 100 LBS

TABLE V
STRAIN READINGS LOADING CONDITION D

	GAUGE NUMBER	Computed μ in./in.	Measured μ in./in.	$\frac{\text{Computed}}{\text{Measured}}$
Center Floor Beam	I-1	424	397	1.07
	I-2	427	439	0.97
	I-3	378	389	0.97
	I-4	275	271	1.01
	I-5	178	160	1.11
	I-6	*	*	-
	I-7	*	*	-
Section 1-1 (Ribs)	II-3	341	293	1.16
	II-6	844	760	1.11
	II-8	832	731	1.14
	II-9	260	220	1.18
	II-10	192	159	1.21

* Too small and within the range of tolerance of recording instruments.

P = 100 LBS

TABLE VI
STRAIN READINGS LOADING CONDITION E

	GAUGE NUMBER	Computed μ in./in.	Measured μ in./in.	Computed Measured
Center Floor Beam	I-1	435	395	1.10
	I-2	558	514	1.09
	I-3	495	478	1.04
	I-4	451	431	1.05
	I-5	424	380	1.12
	I-6	206	165	1.25
	I-7	*	*	-
Section 1-1 (Ribs)	II-3	185	152	1.22
	II-6	196	181	1.08
	II-8	205	175	1.17
	II-9	209	158	1.32
	II-10	*	*	-

* Too small and within the range of tolerance of recording instruments.

P = 100 LBS.

TABLE VII
STRAIN READINGS LOADING CONDITION E₁

	GAUGE NUMBER	Computed μ in./in.	Measured μ in./in.	Computed Measured
Center Floor Beam	I-1	435	380	1.14
	I-2	331	294	1.13
	I-3	243	195	1.25
	I-4	169	138	1.22
	I-5	*	*	-
	I-6	*	*	-
	I-7	*	*	-
Section 1-1 (Ribs)	II-3	185	137	1.35
	II-6	170	152	1.12
	II-8	153	135	1.13
	II-9	135	112	1.20
	II-10	*	*	-

* Too small and within the range of tolerance of recording instruments.

P = 100 LBS

TABLE VIII
STRAIN READINGS LOADING CONDITION F

	GAUGE NUMBER	Computed μ in./in.	Measured μ in./in.	$\frac{\text{Computed}}{\text{Measured}}$
Center Floor Beam	I-1	317	265	1.20
	I-2	388	346	1.12
	I-3	349	331	1.05
	I-4	313	292	1.07
	I-5	282	243	1.16
	I-6	*	*	-
	I-7	*	*	-
Section 1-1 (Ribs)	II-3	309	357	0.87
	II-6	1343	975	1.38**
	II-8	337	412	0.82
	II-9	339	382	0.89
	II-10	1349	907	1.49**

* Too small and within the range of tolerance of recording instruments.

** Load directly over rib.

P = 100 LBS

TABLE IX
STRAIN READINGS LOADING CONDITION F_1

	GAUGE NUMBER	Computed	Measured	Computed
		μ in./in.	μ in./in.	$\frac{\text{Computed}}{\text{Measured}}$
Center Floor Beam	I-1	317	253	1.25
	I-2	250	206	1.21
	I-3	191	146	1.31
	I-4	*	*	-
	I-5	*	*	-
	I-6	*	*	-
	I-7	*	*	-
Section 1-1 (Ribs)	II-3	309	331	0.93
	II-6	197	180	1.09
	II-8	169	140	1.21
	II-9	147	112	1.31
	II-10	*	*	-

* Too small and within the range of tolerance of recording instruments.

P = 100 LBS

APPENDIX I
COMPUTER PROGRAM LISTINGS

I(a) PROGRAM USED TO DETERMINE THE TORSIONAL RIGIDITY

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1 REM THIS PROGRAM FINDS THE TORSIONAL RIGIDITY FOR ORTHOTRO-
2 REM PIC DECKS WITH TRAPEZOIDAL RIBS
3 REM T1=THICKNESS OF THE DECK PLATE
4 REM T2=THICKNESS OF THE RIB PLATE
5 REM A1=WIDTH OF THE RIB (A)
6 REM E1=SPACING OF THE RIBS (E)
7 REM H1=DEPTH OF A RIB(DO NOT INCLUDE DECK)
8 REM H2=LENGTH OF ONE SIDE OF A RIB
9 REM B=WIDTH OF THE BASE OF THE RIB
10 REM E=MODULUS OF ELASTICITY
11 REM G=MODULUS OF RIGIDITY
12 REM S=ACTUAL SPACING OF FLOOR BEAMS
15 READ T1,T2,A1,E1,H1,H2,B,E,G,S
20 U=2*H2+B
30 A=((A1+B)/2)*H1
40 K=(4*A^2)/((U/T2)+(A1/T1))
42 P=G*K
44 PRINT "THE TORSIONAL STIFFNESS IS (K-IN^2)*P
50 E2=E*(T1)^3/10.98
60 R1=(T2/T1)^3
70 S2=.51*S
80 L1=(2*A1+B)*(A1+E1)*B*H2-(R1*A1^3*(E1-B))
90 L2=8*H2*(A1^2+A1*B+B^2)+B^3+R1*A1^3
100 L=L1/((A1+B)*L2)
110 C1=(L/2)*(B/A1)
120 C2=(L/2)*((A1-B)/A1)-((A1+E1)/(A1+B))*(B/(2*A1))
130 V1=(G*K/E2)*(A1^3/(12*(A1+E1)^2))*(3.1416/S2)^2
140 V2=(E1/A1)^3+((E1-B)/(A1+B)+L)^2
150 V3=(L^2/R1)*(B/A1)^3+(24/R1)*(H2/A1)*(C1^2+C1*C2+C2^2/3)
160 V=1+V1*(V2+V3)
170 W=1/V
172 PRINT "THE TORSIONAL REDUCTION FACTOR IS"V
180 H=0.5*((V*G*K)/(A1+E1))
182 PRINT"THE TORSIONAL RIGIDITY IS (K-IN^2/IN)*H
300 DATA .125,.061,3.,3.,3.,3.03,1.5,394,145.93,36.
999 END

```

READY
RUN

AX 14:50PDT 07/23/73

THE TORSIONAL STIFFNESS IS (K-IN^2) 179.781
 THE TORSIONAL REDUCTION FACTOR IS 0.168142
 THE TORSIONAL RIGIDITY IS (K-IN^2/IN) 2.51906

I(b) PROGRAM USED TO COMPUTE DEFLECTIONS AND MOMENTS

PROB

LOAD CONDITION A ONE WHEEL LOAD IN CENTER OF DECK LOAD= 8.1 KIP

TABLE 1. CONTROL DATA

	TABLE NUMBER					
	2	3	4	5	6	7
HOLD FROM PRECEDING PROBLEM (I=HOLD)	-0	-0	-0	-0	-0	-0
NUM CARDS INPUT THIS PROBLEM	1	14	0	1	0	1

TABLE 2. CONSTANTS

NUMBER OF INCREMENTS IN X DIRECTION MX	12
NUMBER OF INCREMENTS IN Y DIRECTION MY	24
INCREMENT LENGTH IN X DIRECTION HX	6.000E+00
INCREMENT LENGTH IN Y DIRECTION HY	6.000E+00
POISSONS RATIO	-0.
SLAB THICKNESS (IF BLANK, MAX PRINCIPAL MOMENT IS COMPUTED -- IF SPECIFIED, MAX PRINCIPAL STRESS)	-0.

TABLE 3. JOINT STIFFNESS AND LOAD DATA

FROM JOINT	THRU JOINT	OX	OY	FX	FY	O	S
0	0 12 24	7.310E-02	9.193E+01	-0.	-0.	-0.	-0.
0	0 12 0	-3.658E-02	-4.596E+01	-0.	-0.	-0.	-0.
0	24 12 24	-3.658E-02	-4.596E+01	-0.	-0.	-0.	-0.
0	6 12 6	-0.	-0.	1.027E+04	-0.	-0.	-0.
0	12 12 12	-0.	-0.	1.027E+04	-0.	-0.	-0.
0	18 12 18	-0.	-0.	1.027E+04	-0.	-0.	-0.
0	0 12 0	-0.	-0.	1.075E+04	-0.	-0.	-0.
0	24 12 24	-0.	-0.	1.075E+04	-0.	-0.	-0.
0	0 0 24	-0.	-0.	-0.	1.747E+04	-0.	-0.
12	0 12 24	-0.	-0.	-0.	1.747E+04	-0.	-0.
0	0 0 0	-0.	-0.	-0.	-0.	-0.	5.000E+02
12	0 12 0	-0.	-0.	-0.	-0.	-0.	5.000E+02
0	24 0 24	-0.	-0.	-0.	-0.	-0.	5.000E+02
12	24 12 24	-0.	-0.	-0.	-0.	-0.	5.000E+02

TABLE 4. JOINT STIFFNESS AND LOAD DATA CONTO

FROM JOINT	THRU JOINT	RX	RY	TX	TY
NONE					

TABLE 5. MESH STIFFNESS DATA

FROM MESH	THRU MESH	C
1	1 12 24	2.519E+00

TABLE 6. BAR STIFFNESS DATA

FROM BAR	THRU BAR	PX	PY	PBX	PBY
-------------	-------------	----	----	-----	-----

NONE

TABLE 7. MULTIPLE LOAD DATA

FROM JOINT	THRU JOINT	QM
6 12 6 12		-1.000E-01

PROGRAM SLAB40 - DEL DECK - ENDRES - REVISION DATE 05 AUG 69
 H.REXIN ORTHOTROPIC DECK MAY 1973
 PLASTIC ORTHOTROPIC DECK

I-----TRIM

PROB (CONT'D)

LOAD CONDITION A ONE WHEEL LOAD IN CENTER OF DECK LOAD= 6.1 KIP

TABLE 8. RESULTS

SLAB MOMENTS ARE PER UNIT WIDTH
 BEAM MOMENTS ARE TOTAL PER BEAM
 X TWISTING MOMENT = - Y TWISTING MOMENT
 - BETA ANGLES ARE CLOCKWISE

X, Y	DEFL	SLAB X MOMENT	SLAB Y MOMENT	SLAB X TWISTING MOMENT	LARGEST PRINCIPAL SLAB MOMENT	BETA X TO LARGEST MOMENT	SUPPORT REACTION
		BEAM X MOMENT	BEAM Y MOMENT				
0 24	-5.080E-05	-2.116E-20 -6.215E-19	8.506E-16 3.232E-13	8.254E-05	8.254E-05	-45.0	2.500E-02
1 24	-2.365E-04	2.290E-08 6.726E-03	1.418E-16 0.	3.133E-06	3.144E-06	-44.9	0.
2 24	-4.006E-04	3.458E-08 1.016E-02	7.088E-16 0.	2.191E-06	2.209E-06	-44.8	0.
3 24	-5.306E-04	3.866E-08 1.135E-02	1.418E-16 0.	9.182E-07	9.377E-07	-44.4	0.
4 24	-6.226E-04	3.849E-08 1.133E-02	1.418E-16 0.	-2.609E-07	2.809E-07	42.9	0.
5 24	-6.767E-04	3.695E-08 1.085E-02	2.835E-16 0.	-7.257E-07	7.444E-07	44.3	0.
6 24	-5.944E-04	3.614E-08 1.061E-02	2.835E-16 0.	3.334E-14	3.614E-08	-0.0	0.
7 24	-6.767E-04	3.695E-08 1.085E-02	0.	7.257E-07	7.444E-07	-44.3	0.
8 24	-6.226E-04	3.849E-08 1.133E-02	1.418E-16 0.	2.609E-07	2.809E-07	-42.9	0.
9 24	-5.306E-04	3.866E-08 1.135E-02	2.835E-16 0.	-9.182E-07	9.377E-07	44.4	0.
10 24	-4.006E-04	3.458E-08 1.016E-02	0.	-2.191E-06	2.209E-06	44.8	0.
11 24	-2.365E-04	2.290E-08 6.726E-03	1.418E-16 0.	-3.133E-06	3.144E-06	44.9	0.

APPENDIX II
STIFFNESS PROPERTIES OF MODEL

IDEALIZED SECTION

	AREA (in ²)	DISTANCE X (in)	AX (in ³)	AX ² (in ⁴)	I (in ⁴)	I+AX ² (in ⁴)
DECK PLATE	3.114	0.0625	0.195	0.012	0.004	0.016
WEB	0.488	4.1250	2.013	8.304	2.603	10.907
BOTTOM FLANGE	0.313	8.187	2.558	20.948	0.000	20.948
Σ	3.915	$\frac{4.766}{3.915} = 1.217$	4.766	-	-	31.871

$$\begin{aligned}
 I_{na} &= 31.871 - (3.915)(1.217)^2 \\
 &= 31.871 - 5.798 \\
 I_{na} &= 26.073 \text{ in}^4
 \end{aligned}$$

$$\text{FLOOR BEAM STIFFNESS } EI = (394)(26.073) = 10.272 \times 10^3 \text{ k-in}^2$$

FIG. II-2 INTERIOR FLOOR BEAM STIFFNESS

	AREA (in ²)	DISTANCE X (in)	Ax (in ³)	Ax ² (in ⁴)	I (in ⁴)	I + Ax ² (in ⁴)
DECK PLATE	1.556	0.0625	0.0973	0.0061	0.0020	0.0081
WEB	1.000	4.1250	4.1250	17.0156	5.3333	22.3489
FLANGE	0.312	8.187	2.5585	20.9484	0.0004	20.9488
Σ	2.868	$\frac{6.781}{2.868} = 2.364$	6.781	-	-	43.306

$$\begin{aligned}
 I_{na} &= 43.306 - (2.868)(2.364)^2 \\
 &= 43.306 - 16.027 \\
 I_{na} &= 27.28 \text{ in}^4
 \end{aligned}$$

$$\text{FLOOR BEAM STIFFNESS } EI = (394)(27.28) = 10.748 \cdot 10^3 \text{ k-in}^2$$

FIG. II-3 EXTERIOR FLOOR BEAM STIFFNESS

IDEALIZED SECTION

	AREA (in ²)	DISTANCE X (in)	Ax (in ³)	Ax ² (in ⁴)	I (in ⁴)	I + Ax ² (in ⁴)
DECK PLATE	0.799	0.0625	0.0499	0.0031	0.0010	0.0041
RIB WEBS	0.369	1.595	0.5896	0.9404	0.2581	1.1985
RIB BOTTOM	0.091	3.095	0.2816	0.8716	0.0000	0.8716
Σ	1.259	$\frac{0.921}{1.259} = 0.731$	0.921	-	-	2.074

$$I_{na} = 2.074 - (1.259)(0.731)^2$$

$$= 2.074 - 0.674$$

$$I_{na} = 1.40 \text{ in}^4$$

$$\text{STIFFNESS } EI = (394)(1.40) = 551.6 \text{ k-in}^2$$

$$D_y = \frac{551.6}{6} = 91.93 \frac{\text{k-in}^2}{\text{in}}$$

$$D_x = \frac{EI^3}{12(1-\nu^2)} = \frac{(394)(.125)^3}{12(1-.35^2)} = \frac{0.7695}{10.53}$$

$$D_x = 0.0731 \frac{\text{k-in}^2}{\text{in}}$$

FIG. II-4 DECK STIFFNESS