FIELD TESTING OF CONCRETE SLAB AND GIRDER BRIDGES

by

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Structural Model Study of Concrete Slab and
Girder Spans

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P R E F A C E

This report is the second in a series which summarizes a detailed investigation of the behavior of pan-formed concrete slab and girder bridge systems, which are widely used by the Texas Highway Department. The initial report treated the detailed techniques developed for the utilization of reduced scale models and also reported on the degree of correlation between the model tests and the full scale prototype testing. This report treats the techniques employed and the results obtained in the field testing of the full scale prototype bridge. The subsequent final report will treat the general behavior and recommendations based thereon from the main model test series.

This work is a part of Research Contract 3-5-66-94, entitled "Structural Model Study of Concrete Slab and Girder Spans." The studies described herein were conducted as a part of the overall research program at The University of Texas at Austin, Center for Highway Research, under the administrative direction of Dean John J. McKetta. The work was sponsored jointly by the Texas Highway Department and the U. S. Department ment of Transportation Federal Highway Administration, under agreement between The University of Texas at Austin and the Texas Highway Department.

Liaison with the Texas Highway Department was maintained through the contact representatives, Mr. L. G. Walker and Mr. B. R. Winn. Mr. I. C. Daniel was the contact representative for the Bureau of Public Roads. Particular thanks are due all of these contact representatives as well as Mr. H. D. Butler, Design Engineer, Mr. C. D. Hanley, Mr. E. L. Hardeman, and Mr. J. Garrett of District 9 of the Texas Highway Department, who were of invaluable assistance during the field testing phase of the project. A special acknowledgment must be made of the excellent cooperation of the R. T. Farr Company, the bridge subcontractor for the field test specimen.

This study was directed by John E. Breen, Professor of Civil Engineering. The model study phase was supervised by E. V. Leyendecker and the field study phase by T. A. Armstrong, both Research Engineers, Center for Highway Research.

A B S T R A C T

This report presents detailed results of a research program to investigate the feasibility of load distribution testing of a full scale reinforced concrete bridge structure in the field and to correlate that data with the data obtained from testing of an accurate scale model. The investigation tested the feasibility and durability of a new type of field strain gage application, tried a new method for field deflection measurement, and evaluated general operations in a remote location. The results obtained from multiple loadings of the full scale structure were compared with results from a 5.5-scale microconcrete model tested in a laboratory. The results were also compared with similar laboratory tested models differing only in skew angle and design loading.

Because of expenses of both field testing and comparison model testing, only one full scale structure was tested. To ensure accurate results, extensive pretesting was done to eliminate faulty materials or methods. This was especially true in the area of strain gage application and testing. Test beams were designed, cast, and tested to destruction to determine the accuracy and durability of the strain gages. As much equipment as possible was shop fabricated for easy field assembly to minimized wastage of time and money at the remote site.

The results of the tests were also compared with existing AASHO design specifications. Recommendations are made for revisions in design criteria, and improvements in both model and full scale field testing.

S U M M A R Y

This report presents a detailed comparison of the loads distributed to various girders of a pan-formed concrete slab and girder bridge and the accuracy of using small scale laboratory models to determine what happens in a full scale bridge structure. A reinforced concrete bridge being constructed in the Central Texas area was instrumented and load-tested to determine the distribution of reinforcement stresses. A very accurate 1/5-scale model was built in the laboratory and instrumented and loaded in a similar manner. The results obtained from the full scale structure and the reduced scale model were compared and exceedingly good correlation was obtained.

During the instrumenting of the full scale structure in the field, a new procedure for applying strain gages and lead-in instrumentation wires to the reinforcement was developed, which worked very well in practice under realistic field conditions. Practically all of the gages placed on the reinforcement were serviceable over a period of several months.

Test results indicated that the AASHO service load distribution factors for moments in longitudinal girders were quite conservative when compared to the measurements developed from both the field test data and the laboratory data. Recommendations are made for further applications of both field and model testing.

I M P L E M E N T A T I O N

This particular study comprised only one portion of Project 3-5-66-94 and, hence, the implementation recommendations are restricted in scope. Based on the results of this program, an improved procedure for obtaining live load strain distribution in field tests is provided and can be utilized by the Texas Highway Department for similar investigations. In addition, the study has shown that the direct reinforced microconcrete modeling techniques, as reported in Research Reports 94-1 and 94-3F, are valid procedures for measuring relative girder load distributions and these procedures can be utilized for similar investigations of the behavior of representative bridge structures at quite a cost savings over large scale testing to destruction. The report does point out that important information concerning service live load behavior can be determined economically in field testing, and this might be preferable where ultimate strength and ultimate safety factor information is not desired.

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INTRODUCTION

1.1 Historical Review

In 1947 the Texas Highway Department realized the need for a more economical standard bridge system for its short span land service road structures. After considerable study, a pan form type of reinforced concrete system with self-supporting metal pans was decided upon. The basic system has been revised many times to increase the range of service. Presently, the pan form bridge is used for a wide range of applications from the farm-to-market roads to the Interstate Highway System.

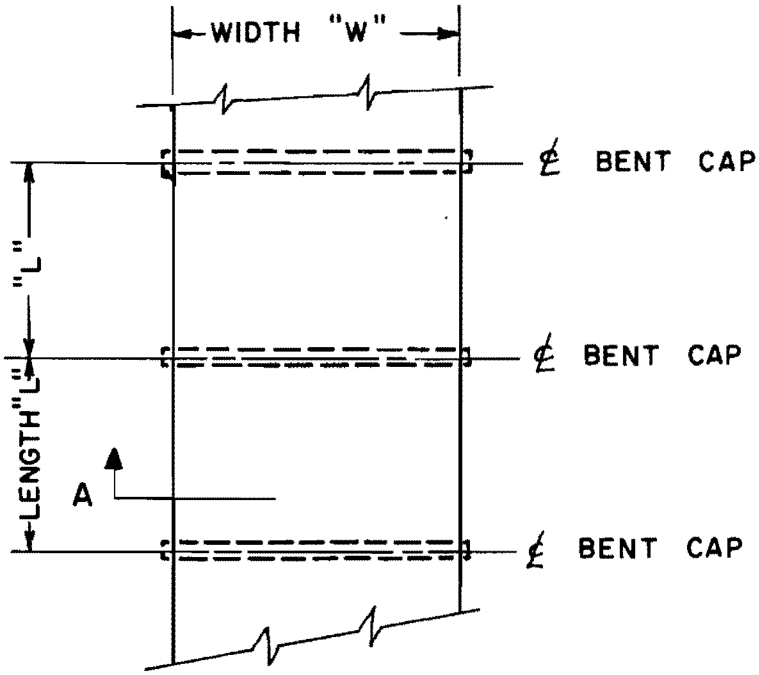
With the expanding usage of this system and subsequent changes in design codes, certain problems and questions have arisen. These are generally in the area of transverse load distribution. Because of the high degree of indeterminancy of the system, an exact stress analysis is very difficult. Only empirical equations and some experimental coefficients have been used to date.

To investigate load distribution characteristics and other factors affecting wider utilization of the bridge system (span length increases and use of high strength steels, for instance), a research program was initiated at the Center for Highway Research at The University of Texas at Austin. The major portion of the project is a reinforced concrete model study of the actual bridge system. As a number of these models were to be tested, the desirability of a correlation study of a full scale bridge became apparent. This report gives a summary of the test of a prototype bridge, the methods used, the problems encountered, and the results obtained.

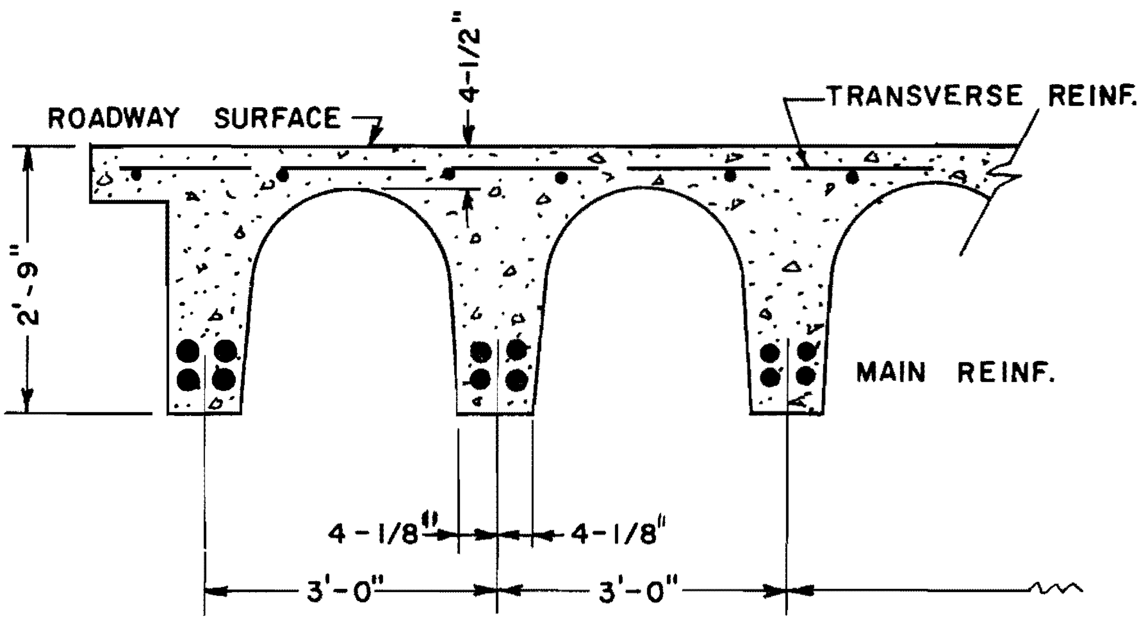
1.2 Description of Bridge System

The Texas Highway Department currently designs and widely contracts for construction a reinforced concrete slab and girder bridge system known as the CGC-type. The slab and girders are cast monolithically on a system of self-supporting steel pan forms. The bridges have a span length of either thirty or forty feet and are supported on reinforced concrete bent caps. Each span is assumed simply supported and each location has as many spans as needed to bridge an open distance. The girders are spaced at three-foot centers and the slab thickness varies with a minimum of four and one-half inches over the crown of each pan (see Fig. 1.1 and Appendix A).

The pan forms for the system are steel and span between clip angles attached to the sides of the bent caps. They are capable of supporting the dead weight of the reinforcement and wet concrete, thus eliminating any need for shoring or falsework. The forms are built into individual units with each pan being three feet wide, similar to the pan joist system of building construction. The bridge mold is formed by bolting each of these pan form units together. The width of the structure can be varied in three-foot increments by adding or eliminating pans.



PLAN VIEW OF BRIDGE SURFACE



PARTIAL SECTION "A"

FIG. 1-1 BRIDGE SYSTEM

The forms are standard sizes and only the reinforcing steel is changed to allow for different magnitudes of loading or different spans. Therefore, each contractor is guaranteed maximimum reuse of his forms, needing only one set of forms for each span length.

Bridges can have different skews relative to the bent caps by sliding the forms relative to each other. Uniform bolt hole spacing permits a number of skew angles from zero degrees to forty-five degrees.

The pan form construction has proven to be a most economical system for its range of span lengths. This method has been adapted for use from the farm-to-market roads to the Interstate Highway System. The advantages of the pan form system are as follows:[5*]

(1) <u>Speed of Construction</u>. Use of pan forms does away with falsework construction and excessive delays.

(2) <u>Light Equipment and Low Labor Requirements</u>. Pan forms can be hauled, erected, and wrecked with a minimum of construction equipment. A smaller construction crew with fewer skilled laborers is required for a pan form job than most other types of construction.

(3) <u>Economy</u>. The concrete pan-formed slab and girder system in this span range will give economy over any other type of construction generally used.

(4) <u>Ease of Widening</u>. In other systems the design is often too inflexible to allow widening of the bridge. Pan form structures can be easily widened by using additional pans.

(5) <u>Little Rubbing Required</u>. Rubbing is not required on surfaces formed with metal forms unless honeycomb is present on the fascia girder.

There are certain disadvantages to this type of construction. They are as follows:[5]

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*Superscript numbers refer to references in the Bibliography.

(1) Excessive Dead Load. The pan form systems are quite heavy when compared with other types of construction. When adequate strata are present near the ground surface (less than thirty feet), this is not serious. When footings or pilings must be founded deep under the surface, cost studies must be made to determine the most economical system.

(2) Bents Must Be Located Accurately. If the bents are too far apart, metal forms will not have sufficient bearing on the temporary end supports. If they are too close together, the forms will foul on the bent caps and cannot be set.

(3) Spans Will Sag with Age. Any nonprestressed concrete girder span will creep with age. This creep is minimized by giving the spans an initial upward camber so that during their average life they will have a satisfactory riding surface.

The design of the bridge was accomplished by considering the transverse slab action and the longitudinal girder action separately. The slab was designed as a one-way slab with the girders as supports. The coefficients for moments in the slab were functions of the relative slab and girder stiffnesses. The girders were designed individually as "T" beams simply supported between the bent caps. Distribution factors have been derived (contrived) for the percentage of load that each girder carries.[9] The loading criterion for lane loading and position of truck was controlled by the American Association of State Highway Officials (AASHO) Specifications for Highway Bridges.[1]

1.3 Leyendecker's Models

As the topic for his doctoral dissertation, Edgar V. Leyendecker constructed models of the pan girder system.[6] They were 1:5.5 microconcrete models. The form work, construction, and load testing of the models was rigidly controlled so as to be as accurate a representation of the prototype as possible.

The forms for the model bridge were replicas of those used on the prototype. The material used was Plexiglas instead of steel. Plexiglas can be molded to the shape needed by heating. Also, it is transparent so that the close tolerances, such as clear cover, can be maintained. During casting of concrete, the forms can be checked for any evidence of honeycombing. Plexiglas also has the property of not bonding to concrete, so the forms need not be oiled prior to casting.

The concrete used is called microconcrete, because the aggregates are scaled to the model size. The specific aggregate sizes are step-graded from the prototype grading. The mix proportions of cement, sand, gravel, and water are the same in both model and prototype.

The reinforcing steel area is modeled by the scale factor. The largest bar used in the prototype was a No. 11 bar, which was scaled to a No. 2 bar on the model. A special order of deformed No. 2 bars was obtained for the tests. The smaller bar sizes were modeled using straightened and annealed wire.

The testing was accomplished by loading the specimen with hydraulic jacks resting on special pads that simulate the wheels of the truck. The rams were placed between the bridge surface and a reaction frame and the load on each jack was increased until the required load was reached. This was equivalent to the design load divided by the square of the scale factor and was controlled by pressure gages and load cells.

Electrical resistance strain gages were attached to the reinforcing steel to give a measure of strain. Strain readings were taken for each loading condition.

Using AASHO trucks,[1] the models were tested under working load, moderate overload, and then loaded to failure. Leyendecker completed testing of four models,, one with a zero degree skew, one with a twenty-six degree skew, and two with forty-five degree skews. They were models of standard forty-foot span bridges.

Leyendecker's testing program had certain objectives, which were:

(1) To investigate the behavior of the typical concrete slab and girder bridge spans at service load, moderate overload, and at ultimate load.

(2) To investigate the feasibility of increased span lengths.

(3) To investigate the behavior of the bridge system using high strength reinforcing steel.

(4) To make recommendations regarding the adequacy of present design provisions based on his test results.

1.4 Objective and Scope

Whenever a model is tested, the basic question which must be raised is "Is this an accurate representation of the prototype?" In the case of Leyendecker's models, verification was important for assessing the credibility of his results. To corroborate the model analysis, a testing program for a full scale bridge span was undertaken. The bridge selected was to be the same length, roadway width, skew angle, and design criterion as one of the models so it would be in the midst of the full spectrum of Leyendecker's data.

The purpose of the full scale test was twofold. First, as mentioned previously, to correlate with and corroborate the model data. Second, to investigate the feasibility of load-distribution testing of a full scale structure in the field. Since the bridge to be tested ultimately will be in actual highway use, loading to destruction was impossible. The possibility of severe overloads also had to be discarded for economic reasons. The service load testing program is an extremely important element, since it gives the clearest information as to load distribution across the section at the range of loads in which the designer is presently most interested.

Economic feasibility is the most important factor in consideration of full scale testing. In ultimate load testing the model is inherently cheaper, both in fabrication and testing costs. The fabrication costs decrease as the scale ratio becomes smaller than unity, due to less materials used and less labor required. However, as the scale factor becomes even smaller, the fabrication costs begin to rise because specialized materials and labor are required. The cost of ultimate load testing is also a function of scale factor; as a model becomes smaller, the load required to destroy the specimen becomes less. In addition, the support and reaction frames are smaller. Clearly, there is a minimum cost or optimum scale factor available. Figure 1.2 shows the plots of percentage cost versus scale factor[3] for ultimate load model tests. However, economics for service load testing vary considerably. Since service load testing of a bridge prototype requires only a loading truck or trucks, the cost of a heavy reaction frame is eliminated. Also the fabrication costs may be lowered considerably, because the prototype is not laboratory built but is part of a construction job which is funded for another purpose. Only the added costs of instrumentation and gaging remain. As shown in Fig. 1.2, for a full scale model the fabrication is sixty percent of the total cost and ultimate loading is forty percent of total cost. Since for this program fabrication and loading costs were

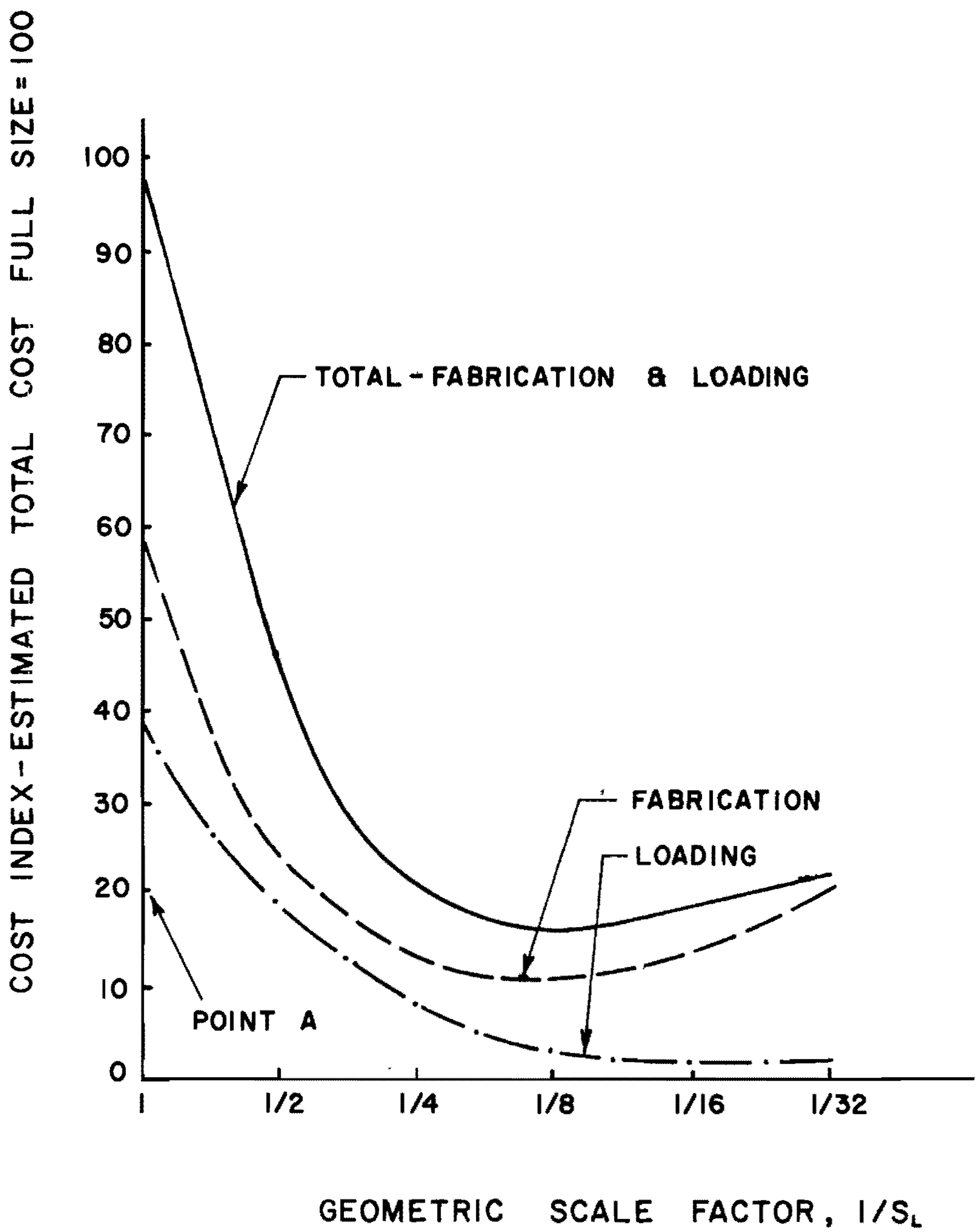


FIG. 1-2 ESTIMATED COST VARIATION WITH SCALE FACTOR REINFORCED CONCRETE BRIDGE MODEL

small in comparison, the overall estimated cost index for the service load testing program was twenty percent of full scale ultimate load tests (point A, Fig. 1.2).

There are other factors that must be considered in a full scale field test. Since the construction of the bridge is under contract to a construction firm, their schedule must be closely followed to prevent costly delays. Field construction techniques are not as refined or rigidly controlled as laboratory work, so care must be exercised to ensure proper construction. Also, construction workers are not always aware of the importance and fragility of some instrumentation (such as strain gages), so they must be cautioned and their actions controlled, so that delicate articles are not ruined by accident.

In summary, the plan of this program was to instrument and test under service load in the field a full scale bridge of the type modeled by Leyendecker. The purpose was to substantiate the model data and investigate the feasibility of full scale field testing.

CHAPTER II

INITIAL STUDIES AND PREPARATION

2.1 Initial Test Beams

To ensure the best possible results in the field, two test beams were cast and tested in the laboratory to check certain unknowns in the system. The initial testing had three purposes:

(1) To check the durability of the strain gage system during casting.

(2) To determine the proper type of strain gage to be used.

(3) To check the feasibility of the proposed strain gage field connection method.

Since field control of construction is not as rigid as in the laboratory, the strain gage system had to be checked to ensure that it was protected against damage (such as vibration of the concrete during casting). The method of applying and protecting the gages used in Leyendecker's model studies had displayed a very high degree of success. This method employs a flexible waterproofing cement to cover the entire gage and a rigid epoxy coating applied over the waterproofing. The method of application and protection will be discussed in Chapter III.

The type of gage used is also critical. It must be durable and easy to apply. Epoxy-backed foil gages with a one-quarter inch gage length have been used in the model series. Three other similar gage types were also investigated for use in the field test series.

To connect the gages to the strain measuring equipment, lead wires must be attached to the strain gage. In the models, holes were drilled in the bottoms of the forms and the gage lead wires were passed through these holes. After removal of the forms, the wires were connected to the measuring equipment. In the field test program the contractor did not want holes drilled in his forms, nor was it desirable to have long lengths of exposed wire present below the bridge between casting and testing. So that later connections could be made, the ends of the lead wires had fiberglass-mounted terminals connected to them. One-inch square aluminun cover plates were attached to the terminal connectors, as shown in Fig. 2.1. The plates were taped to the bottom of the forms so that after form removal the plates could be unscrewed and connection to the lead wires made.

The first beam tested used four different types of gages and gage application methods, the protection method used in the models, and the plate connection system. Figure 2.2 shows the physical properties of the beam and the loading diagram. The beam was reinforced for flexure with one No. 11 bar of intermediate grade steel, the same size as in the bridge girders. One of each of the four types of gages was applied to the bottom of the bar, the position believed to afford maximum protection against damage during concrete vibration. During casting of the concrete, the concrete was vibrated extra thoroughly in the area of the gages to test the durability of the system. High early strength cement was used so the beam could be tested as soon as possible. The beam was loaded in

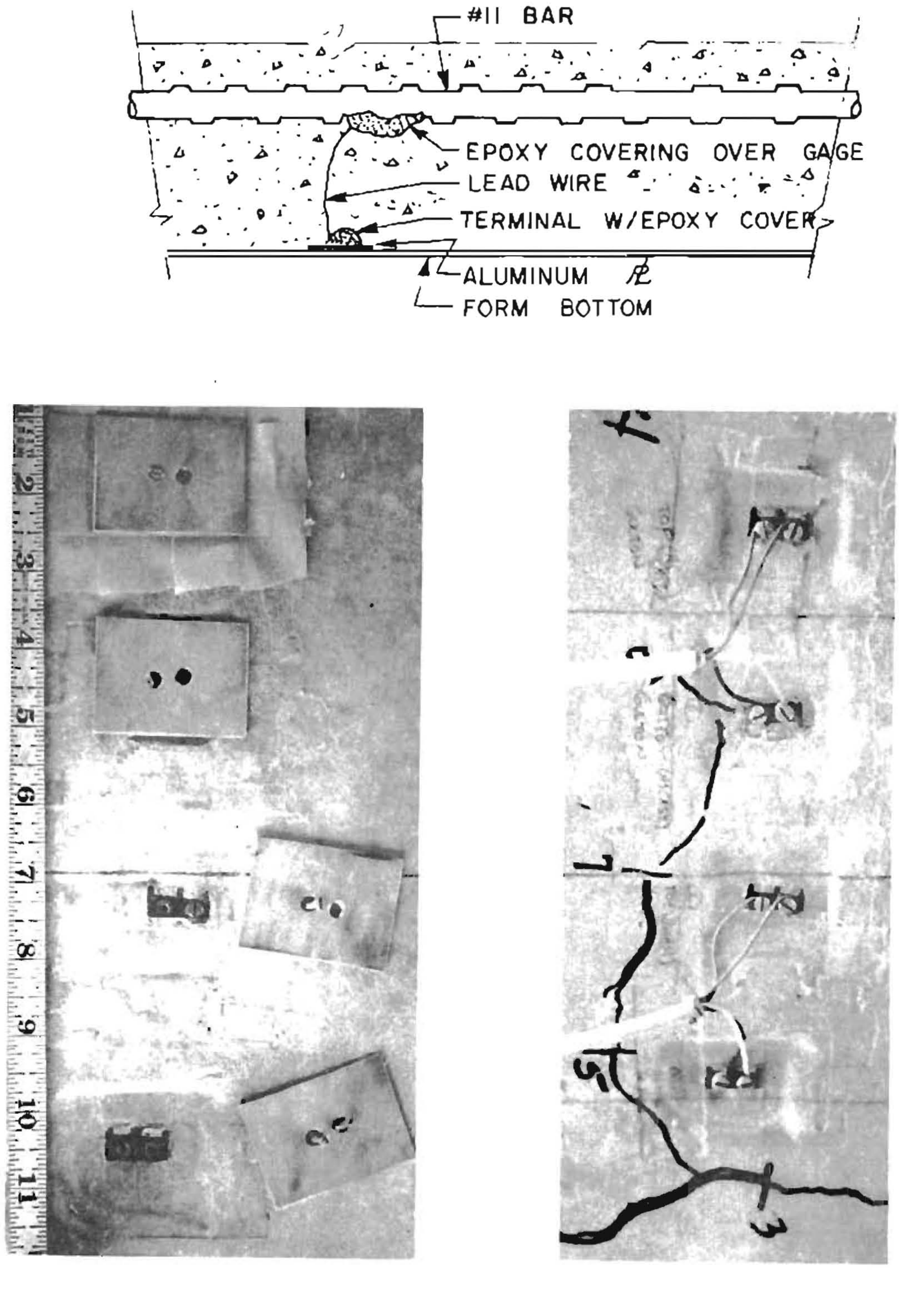


FIG 2-1 TERMINAL & PLATE SYSTEM

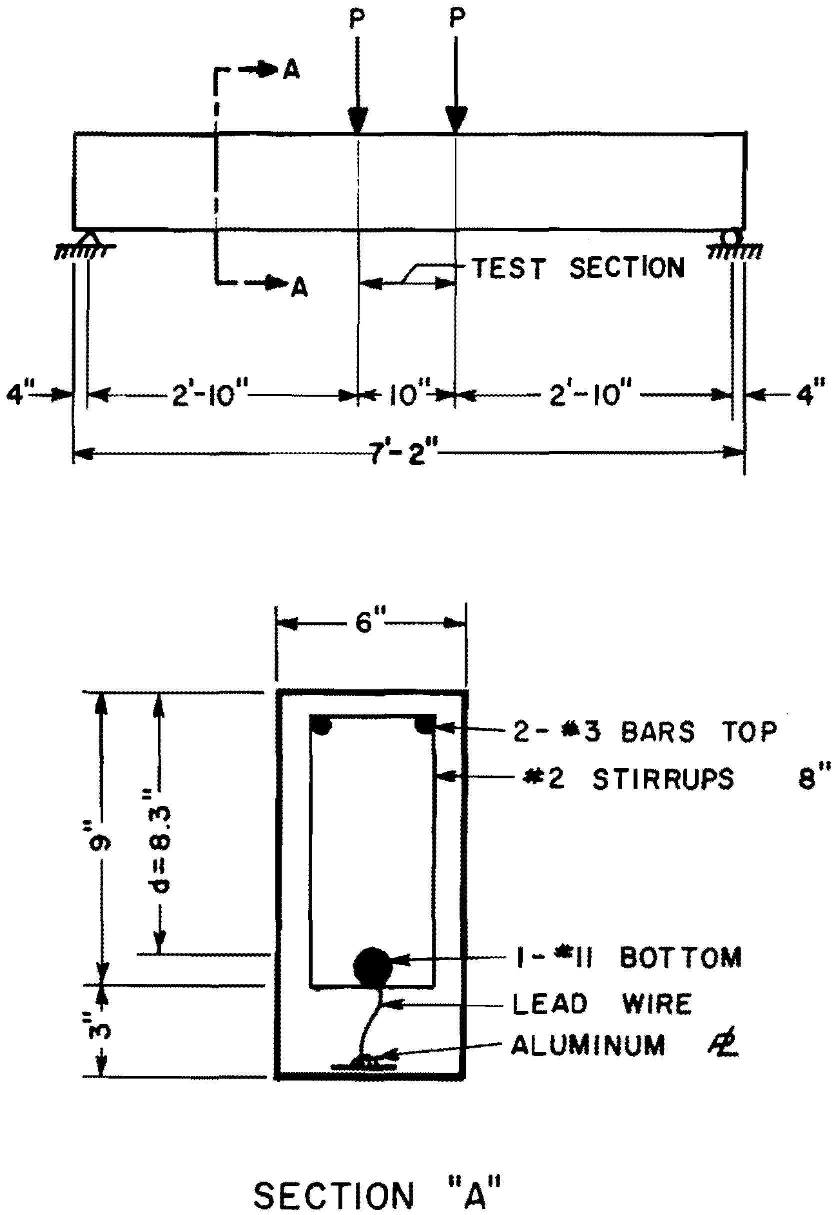


FIG. 2-2 TEST BEAM ONE

a 200,000 pound Young testing machine. Each load stage was in 2,000 pound increments, with strain readings taken at each load stage. The beam was tested to destruction.

The performance of the gage protection and plate connection system was observed during and after casting. These components behaved acceptably and no reason for trouble in the field was seen. However, the decision as to which strain gage type and method of application to use was still in question. Two of the gage systems were discarded as being unsatisfactory. At higher loads they tended to separate from the steel bar and gave erroneous readings.

Another test beam was found necessary to determine the best gage type. A beam similar to the first was used. The dimensions (see Fig. 2.3) were changed to get the same strain gradient in the beam as calculated for the prototype girder at service load levels. This was done to determine if the position of the gage on the bar was of any consequence. The two remaining gage types were applied and connected as in the first test beam. Two gages of each type were used, one on the top and one on the bottom of the bar. The beam was cast as before and all components were thoroughly checked. Testing was accomplished by using a steel reaction frame and hydraulic jacks. The loading procedure was the same as in the previous test beam.

After analysis of the strain data, the epoxy-back foil gage, as used in the model, was judged the most acceptable. Also, the position of the gage on the bar (top or bottom) was found to be of no significance with members of this depth and strains of this magnitude. Therefore, the decision was made to put all strain gages on the bottom of the bar, since that position afforded the most protection. The performance of the

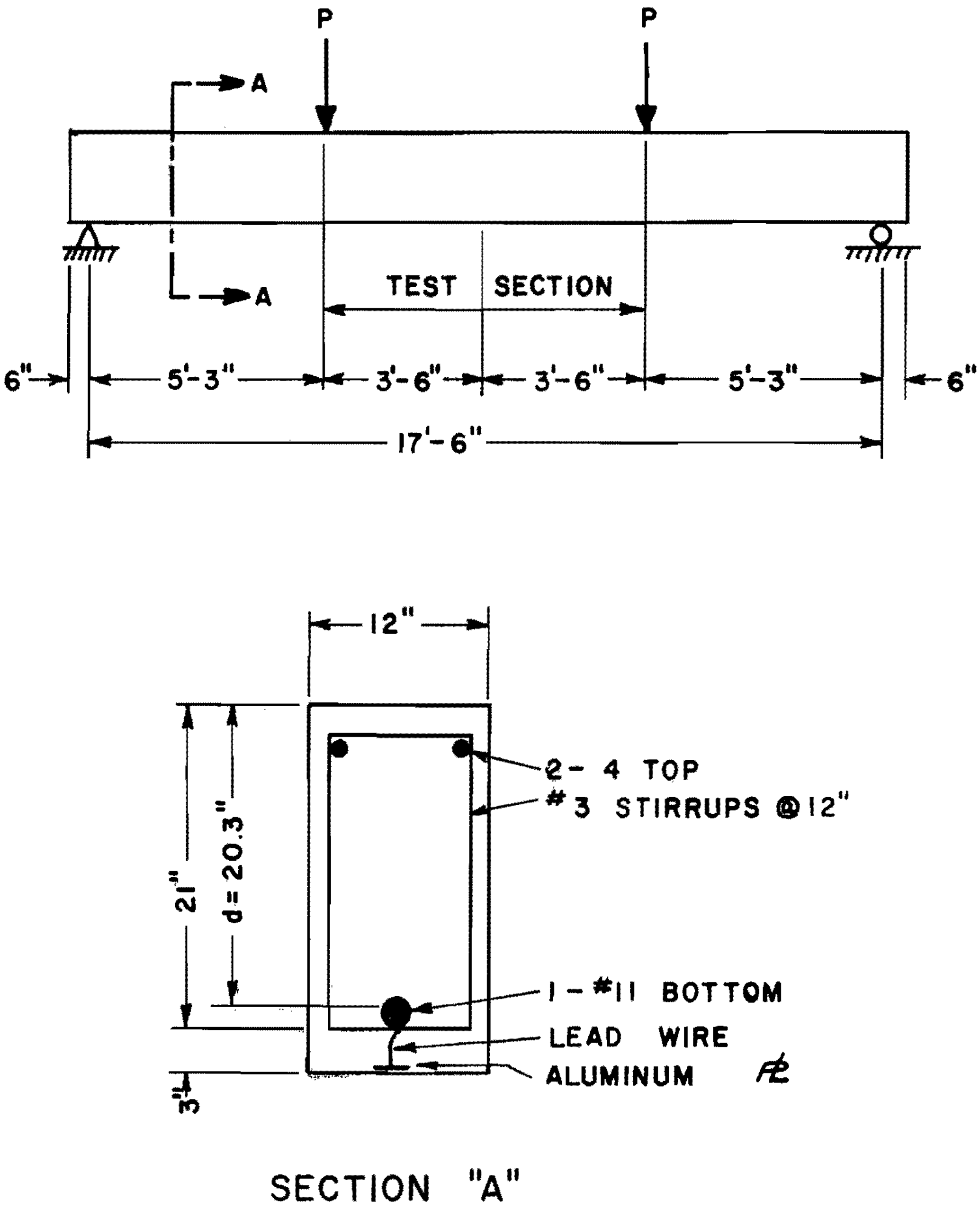


FIG. 2-3 . TEST BEAM TWO

waterproofing and protection system worked very well and showed great promise for field work, since the wiring could be reconnected many times with a minimum of effort.

2.2 Test Bridge

In choosing a test span, certain requirements had to be fulfilled. Primarily, the bridge had to be located close to Austin, Texas, headquarters for the project. Secondly, the construction schedule had to coincide with the progress and timing of the overall project at the University. Thirdly, the span in question had to match closely the length, width, depth, and design loads of the model spans previously tested by Leyendecker.[6,8] This was necessary so the data from his model study of the span could be compared with the data from the rest of the program.

With the cooperation of the Texas Highway Department, a bridge site was chosen that closely agreed with the requirements set. The location for the test bridge was about sixty-five miles north of Austin, between Belton and Nolanville, Texas. The bridge was scheduled for construction about three months after the final test site was chosen. This timing would allow adequate preparation in Austin and at the site prior to casting of the bridge. Although some of the bridge details did not agree exactly with all of the conditions of Leyendecker's initial three models, the length, width, and depth were the same so that the available model forms could be used in construction of the matching model. Due to the nature of service of the highway, the design load was smaller than used on previous models.

The bridge selected has a span length of forty feet, a width of thirty-three feet and a skew angle of twenty-six degrees and thirty-four minutes. Figure 2.4 shows the pertinent information regarding the site and bridge. The test span selected was the second span of a five-span bridge system. This was the most readily accessible from below and had adjacent spans for dead load, as previously modeled by Leyendecker.[6,8]

2.3 Strain Gage Locations

Electrical resistance strain gages, as selected in Section 2.1, were to be located on the reinforcing throughout the test span. The circuitry necessary for testing was limited in capacity and, therefore, limited the number of strain gages that could be used. Leyendecker solved this problem by heavily gaging one-quarter of the model span and using "mirror image" trucks to obtain strains for ungaged portions of the span. The same system was employed on the prototype with one-fourth of the bridge gaged at the center line, quarter point and tenth point of each stem. Certain discrete points outside the main quarter were gaged as checks. For purposes of identifying a point on the bridge, a coordinate system identical to the one used by Leyendecker on his models was adopted. This system letters the girders A to M with I omitted. The transverse direction was numbered at the eighth point of the span. The letters increased from south to north and the numbers increased from west to east. Therefore, a gage located at position C4 would be on the centerline of the third girder from the west. This system made comparisons easy betweeen model and prototype. Figure 2.5 shows the strain gage locations.

The uncertainty of control in casting the bridge raised the question of protection and durability of the gages. Even though laboratory

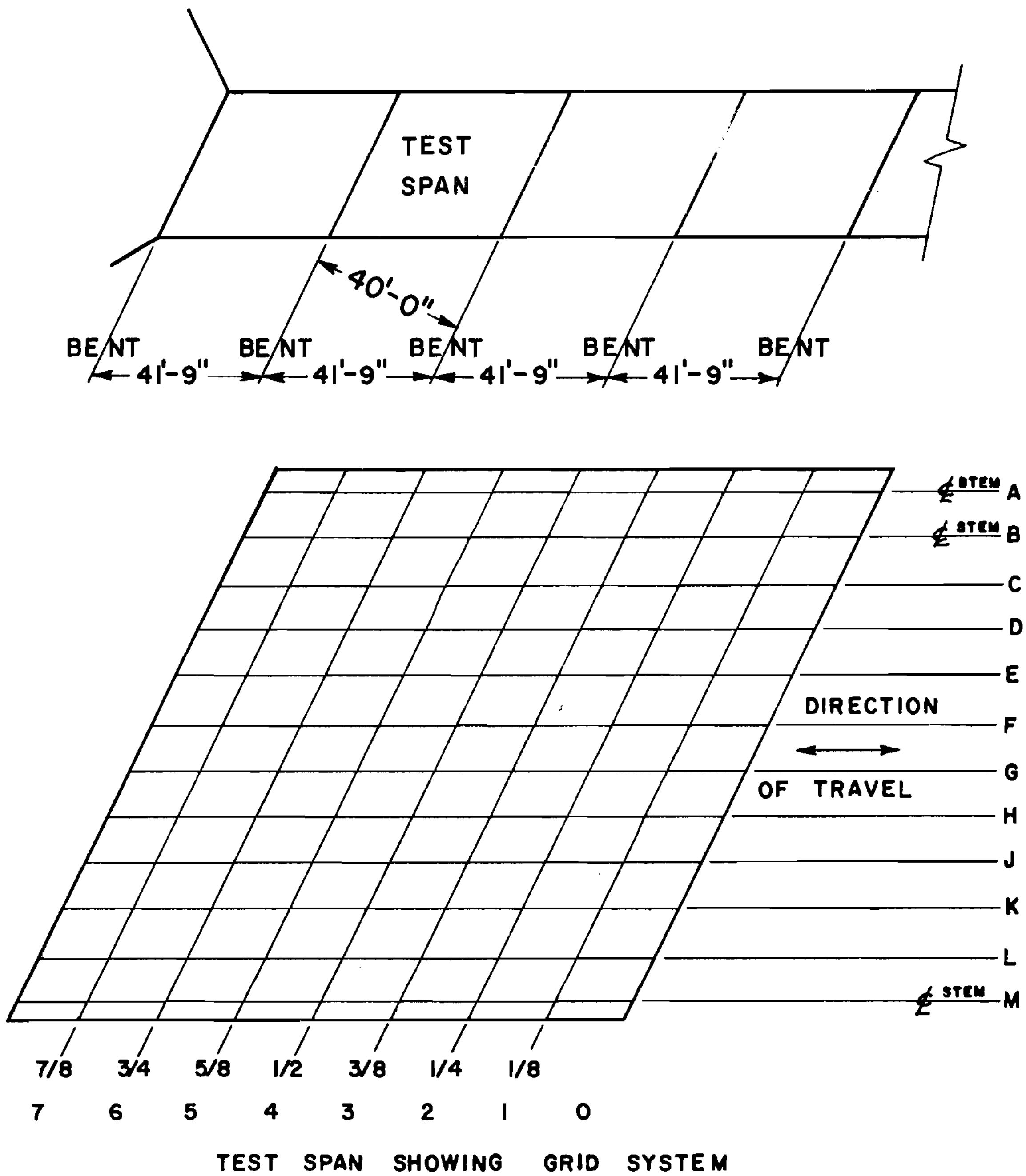


FIG. 2-4 GENERAL PLAN

DEAD LOAD GAGES ON STEMS:

B - 4	B - 2
D - 4	D - 2
F - 4	F - 2

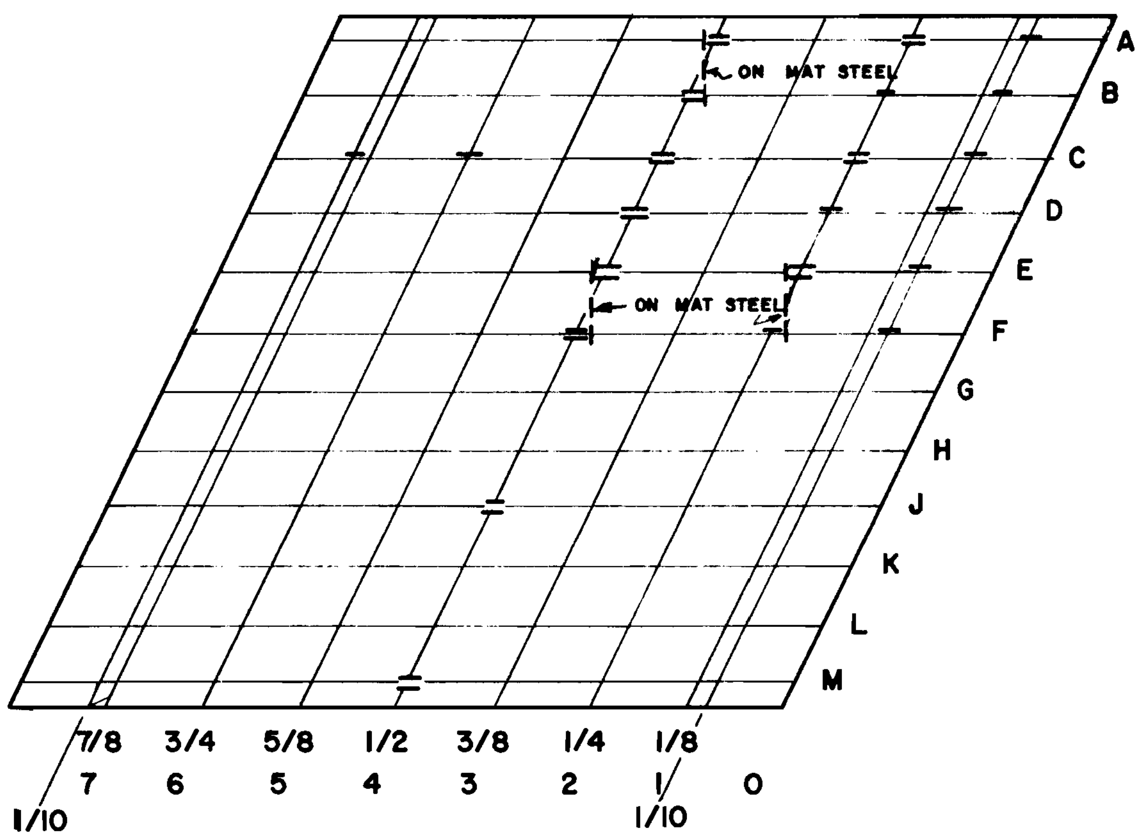


FIG. 2-5 STRAIN GAGE LOCATIONS

tests showed the gage protection system adequate, the chance of any damage had to be minimized in the field. Therefore, certain locations were chosen to have duplicate gages. Since two parallel bars are located on the same level in the bottom of the stems, one gage could be placed on each bar. This extra gage served as a spare in case of damage in casting.

Leyendecker placed strain gages on the transverse mat steel (perpendicular to the direction of the main moment steel) to observe the presence of any transverse strain.[6,8] Gages were placed on steel in the prototype at similar locations, as shown in Fig. 2.5. The bar size in the mat is smaller than the longitudinal steel, but the same size gage was used. No. 5 bars were used in the mat and presented no problem to application of the gages. Leyendecker applied the same gage size to No. 2 bars with no difficulty.[6,8]

Unfortunately, the system of lead wires and plates is only good for recording live load strains, since the connection to the recording circuit can only be made after the removal of the forms. To obtain a measure of dead load strain, certain of the "spare" gages, as indicated in Fig. 2.5, were wired to a switching unit for dead load measurement. The lead wires were threaded through the joints in the pan form system as shown in Fig. 2.6. Due to repeated usage, the pan forms had developed kinks in the bottom flanges that made running the lead wires through the cracks a relatively easy matter.

2.4 Dial Gage System

As a backup instrumentation system in case of failure of the strain gage system, a method for measuring deflections of the bridge was

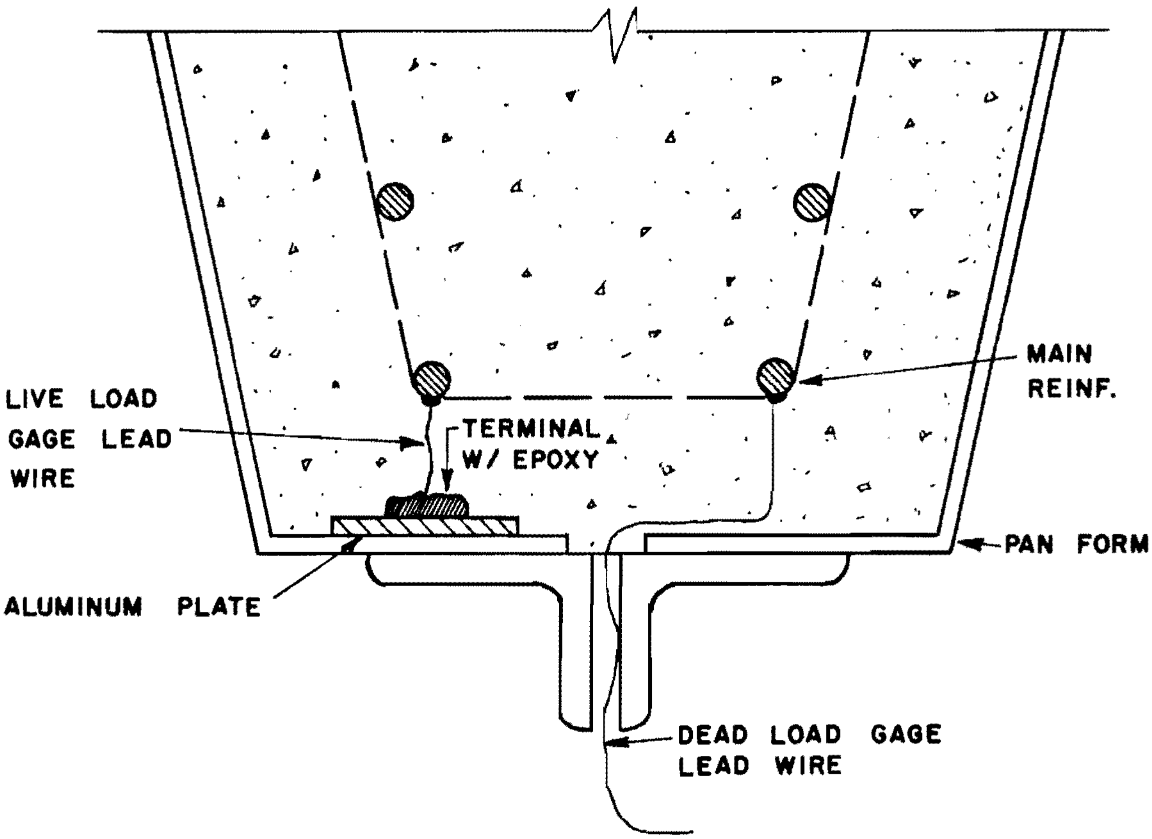


FIG. 2-6 PLATE SYSTEM AND
DEAD LOAD WIRE

devised. After a study of the conditions and needs, a movable under-bridge system using dial gages was chosen. The bent caps had holes with inserts to hold the brackets for the forms. These holes could hold a steel angle which would serve as a track for the movable system. To hold the dial gages a bar joist was designed to bridge the length of the span with cross pieces to hold the dials under several stems. It rested on rollers which ran along the track. Figures 2.7 through 2.9 show details of the dial gage system.

Previous tests on the model indicated that deflections damped quickly as the distance from the load point increased. Therefore, the accuracy of the gages beyond two or three stems is doubtful. The dials have an accuracy to the nearest 0.0005 of an inch and readings in the vicinity of the loads were expected to reach only 0.05 inches. With a limited amount of dial gages available, the system was designed to cover only six stems at a time. However, with the movable system the gages could be centered under any loading area.

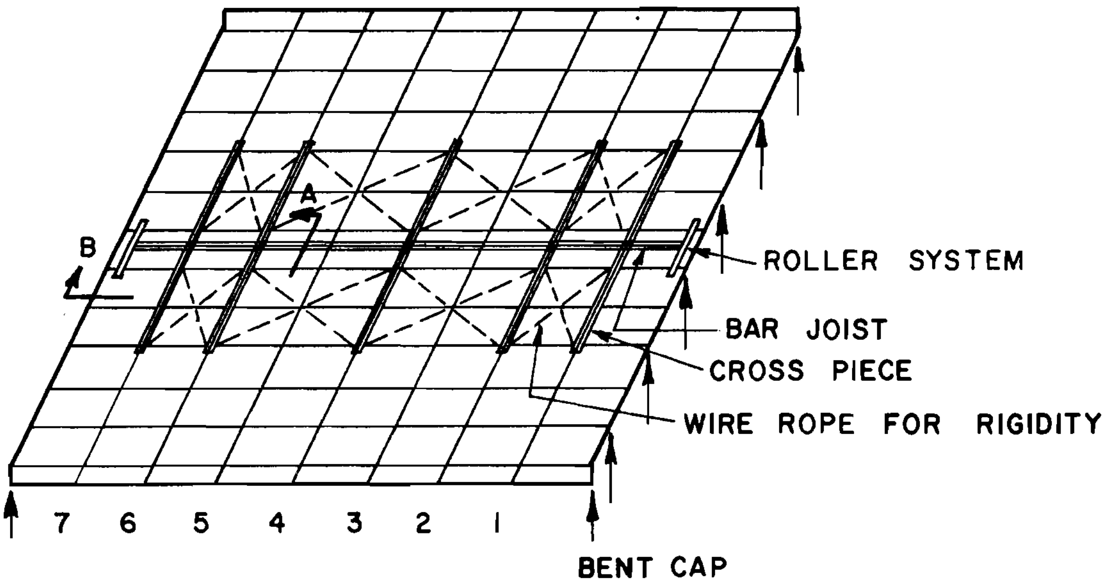
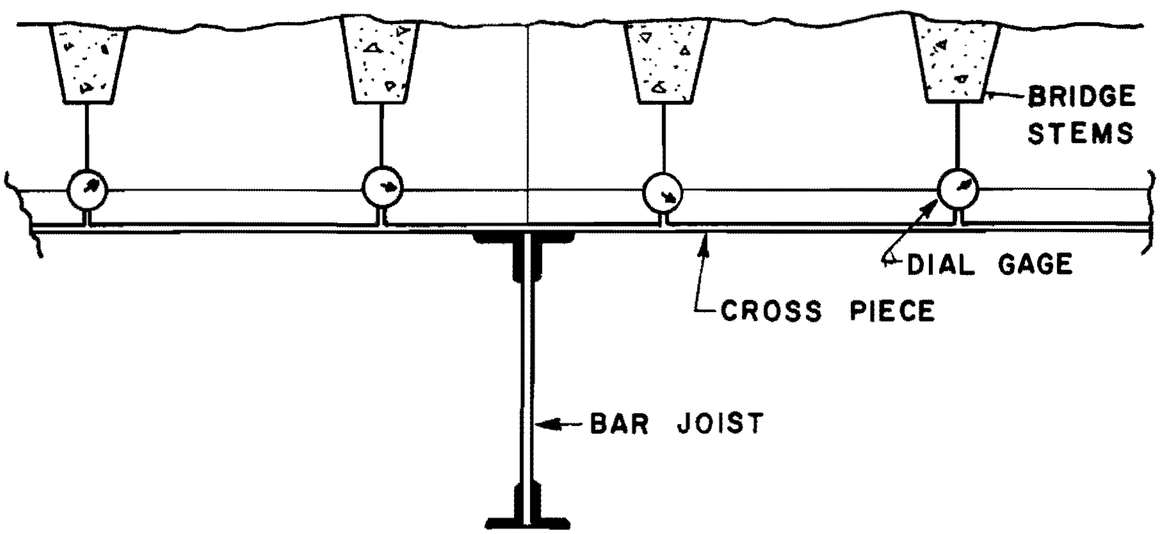
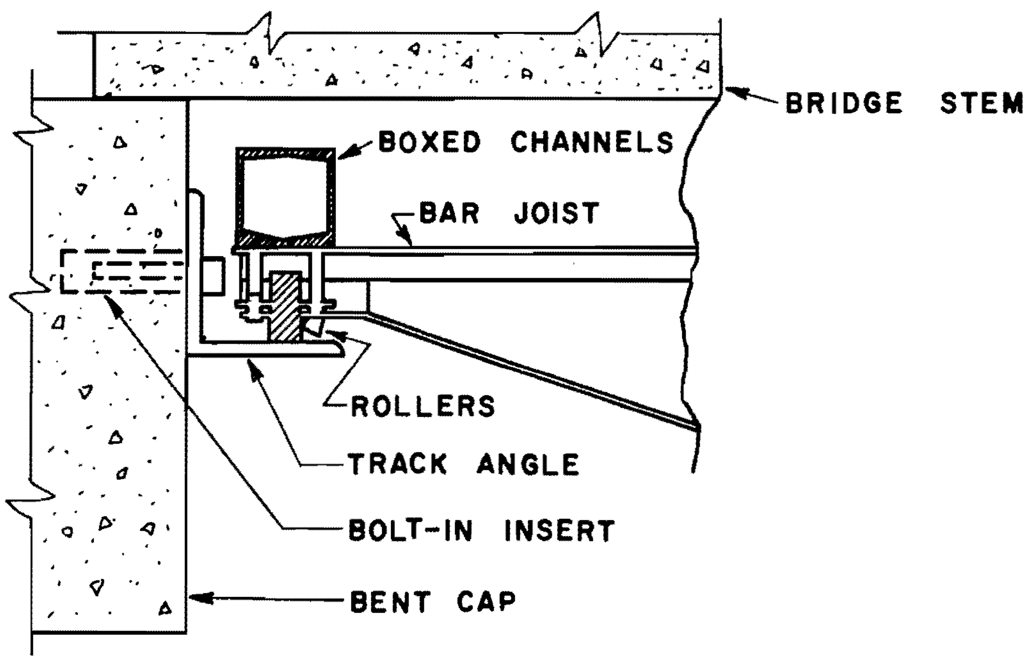


FIG. 2-7 DEFLECTION BRIDGE LAYOUT



SECTION "A"



SECTION "B"

FIG. 2-8 DEFLECTION BRIDGE SECTIONS

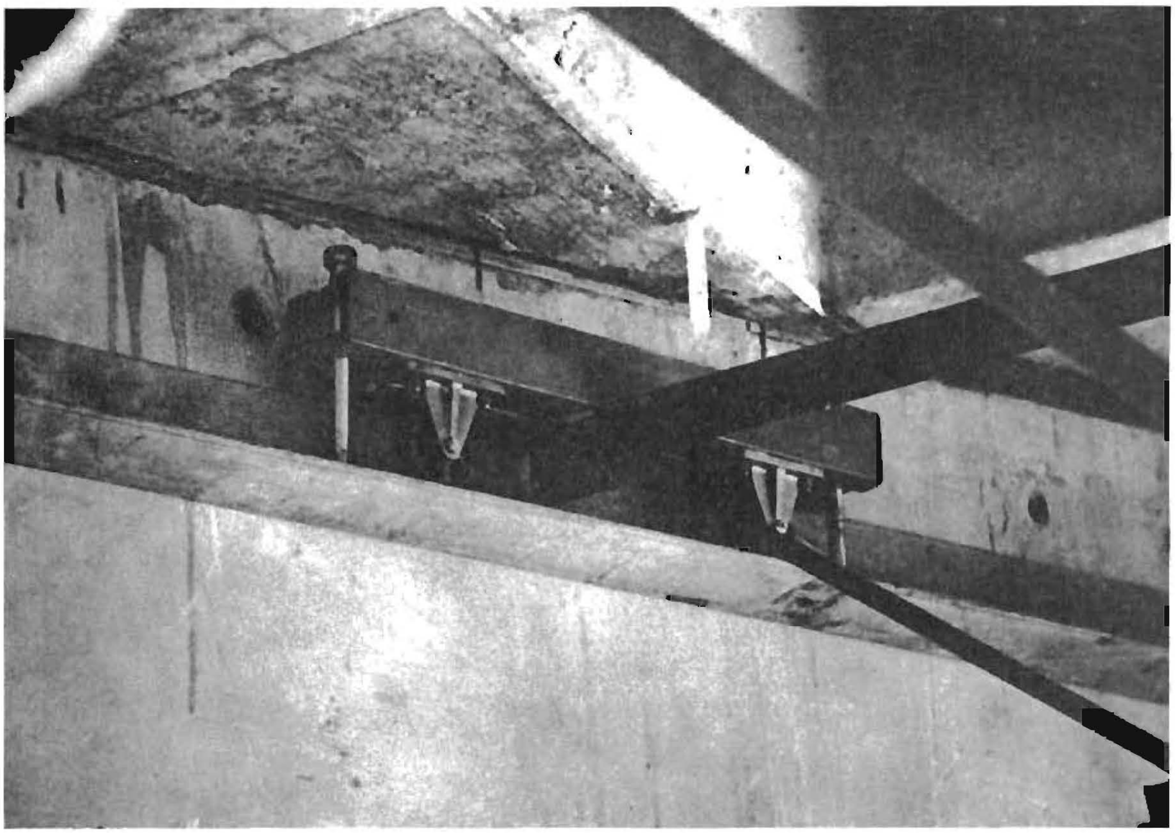


FIG. 2-9 DEFLECTION BRIDGE ROLLER SYSTEM

CHAPTER III

FIELD WORK

3.1 Application of Strain Gages

The first step in preparation for testing was to apply the strain gages to the reinforcing steel. The gages selected were Baldwin SR-4 epoxy-backed foil gages with a one-quarter inch gage length. The entire sequence of strain gage application as used in this program is shown in Fig. 3.1. The steps followed corresponding to Fig. 3.1 were as follows:

Steps 1 and 2. To have a smooth clean surface with room to place the gage, the bar must have some deformations ground off and the bar must be filed and sanded smooth. A hard disc grinder will remove the deformations but leaves a rough surface. The file smooths the surface and sand paper gives the final polish. There can be no pits, burrs, or unevenness on the surface.

Step 3. To remove any filings or dirt that may remain, the polished surface is swabbed with acetone (or other appropriate solvent).

Step 4. So the remaining steps can be done easily, the strain gage is stuck to a piece of clear adhesive tape (the top of the gage on the adhesive part of the tape) and the tape positioned on the bar. The tape is positioned on the bar so the gage is aligned properly along the bar axis. For this test a sight alignment was used. The tape is then peeled back out of the way with one end of the tape still in contact with the bar and the gage still aligned on the tape.

Step 5. With the tape and gage out of the way, the bar is etched with a weak solution of acid to ensure that the gage bonds well when epoxied. The acid is then neutralized and the bar is allowed to dry.

Step 6. So the epoxy glue will dry fast, an accelerating solution is applied to the bar and allowed to dry for about one minute. Then

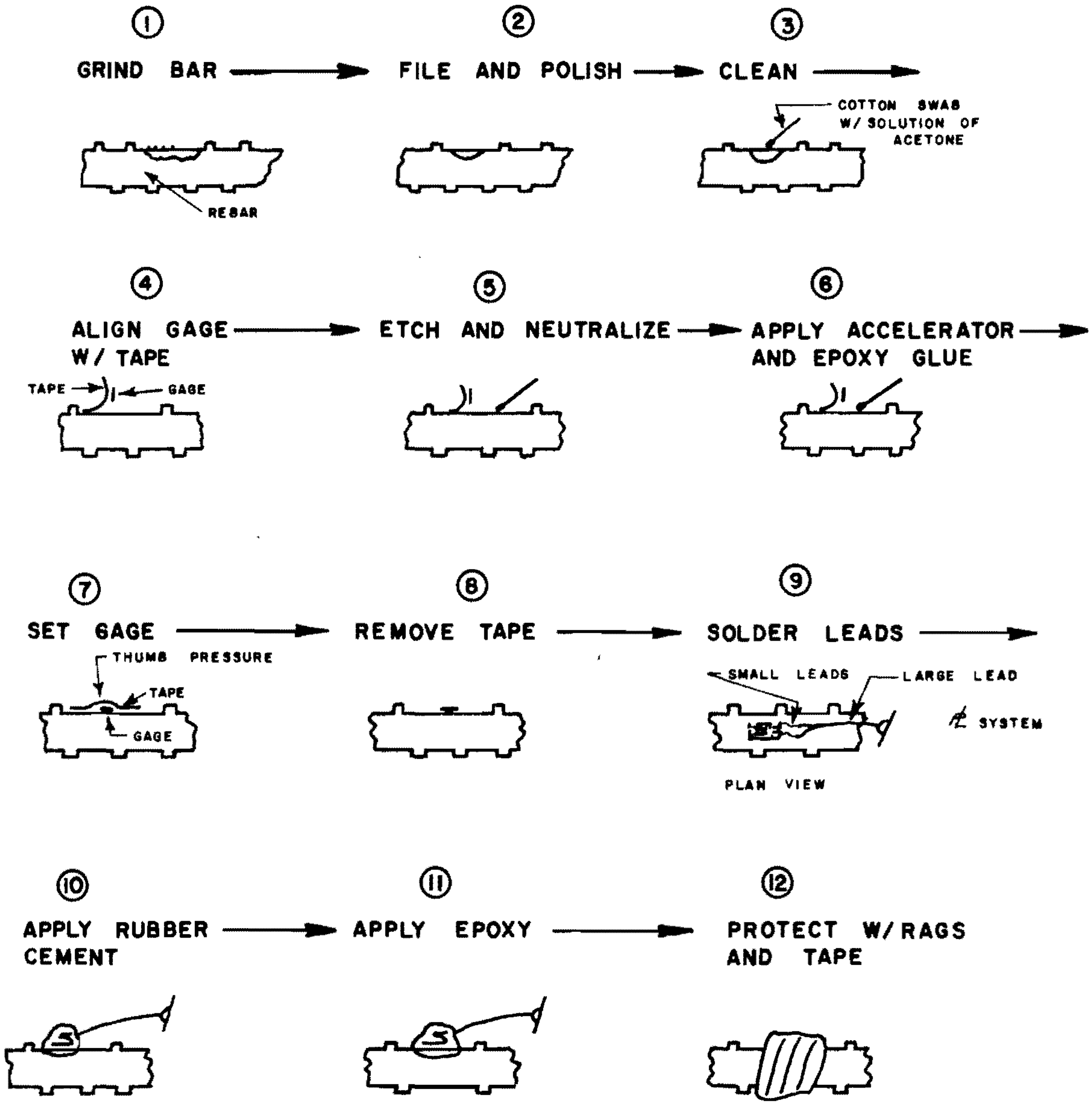


FIG. 3-1 STRAIN GAGE APPLICATION SCHEMATIC

the epoxy glue is applied. The etching, neutralizing, and accelerating solutions are applied with cotton swabs, and the epoxy glue with a small round stick or any object that will apply the epoxy in a thin, even layer.

Step 7. Immediately after the epoxy glue is applied, the tape, with gage, is repositioned on the bar and held in place with thumb pressure for about thirty seconds.

Step 8. After the epoxy glue is completely dry (usually one minute, depending on surrounding conditions) the tape can be removed and the gage will remain fixed to the bar. To ensure no damage has been done to the gage, a continuity check is made at this point. Checks are made from gage terminal to gage terminal and terminals to ground.

Step 9. Next, two small lead wires are soldered* to the gage terminals. These small wires are about one-half inch long and may be difficult to solder to the gages. Care should be taken to ensure that no excess solder gets on the gage proper, as it would short circuit the grid. These small leads are the weak link in the system and are designed so that they will break before the gage in case something goes wrong. Next, the large leads are soldered to the small leads. The large leads are attached to the terminal and plate system described in Chapter II. For this field application, the small loads were soldered to the large leads in the lab.** After another continuity check, the small leads were coated with a high electrical resistance compound to prevent any shorting between the wires.

Step 10. To waterproof the gage and provide a flexible cushion against shearing forces, Devcon rubber cement is applied liberally to the gage and leads.

Step 11. After the cement has dried, an epoxy is applied over the cement to give a hard surface for protection against vibration.

Step 12. Since the reinforcing steel would be left on the job site for some time before use, the gage was further protected by rags wrapped around the bar, a piece of split conduit over the rags, and finally wrapped with rubber tape. The rags, conduit, and tape were removed prior to placement in the forms.

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*Field soldering usually requires higher heat in soldering irons than laboratory work.

**At extra cost gages can be ordered with the small leads, but they seem to offer no advantage or savings over the hand-wired system.

3.2 Construction Phases

The normal sequence of bridge construction was carried out. The reinforcing steel was shop cut, bent, and marked. The beam cages were tied in the normal manner. The only precautions taken were to ensure that the correct instrumented bars were tied, so that the gages would be in the proper sequence and on the bottom of the cage. The chairs to provide clear cover were tied on the cage in the area of the gages to afford as much protection as possible. The cages were lifted and set into the forms using a motor crane. All sequences of this work were performed by the bridge contractor, who cooperated fully and exhibited great care to ensure satisfactory conditions for the test program.

After the gages were set in the forms, the next task was to tape the gage lead connection plates to the form bottoms. The girders are about two and one-half feet deep, so this task was accomplished using staff members with long arms. The six dead load gages had their lead wires pulled through the joints in the forms and connected to a switch and balance unit. The initial reading of the gages was taken so a measure of dead load strain could be obtained after form removal. Taping the gages to the form bottoms and connecting the dead load gages consumed about two hours time with a minimal loss to the contractor's schedule.

The concrete was a standard Texas Highway Department mix and was plant batched.[12] Transmix trucks brought the concrete to the site, where it was transferred to half-yard buckets and lifted to the bridge level with a motor crane. There, the concrete was placed in the forms. The only further protection taken against the concreting operations damaging the gages was hand placement of a shovel of concrete around the gage area to guard against impact. There was no other precaution taken against damage during concrete vibration, except in the area of the gages on the mat steel. Since the mat bars were very close to the surface and, therefore, vulnerable, no direct vibration was permitted in these areas. The care taken in placing the concrete prevented any honeycombing. Twelve test cylinders were taken during casting. Sets of three each were taken from four different concrete trucks.

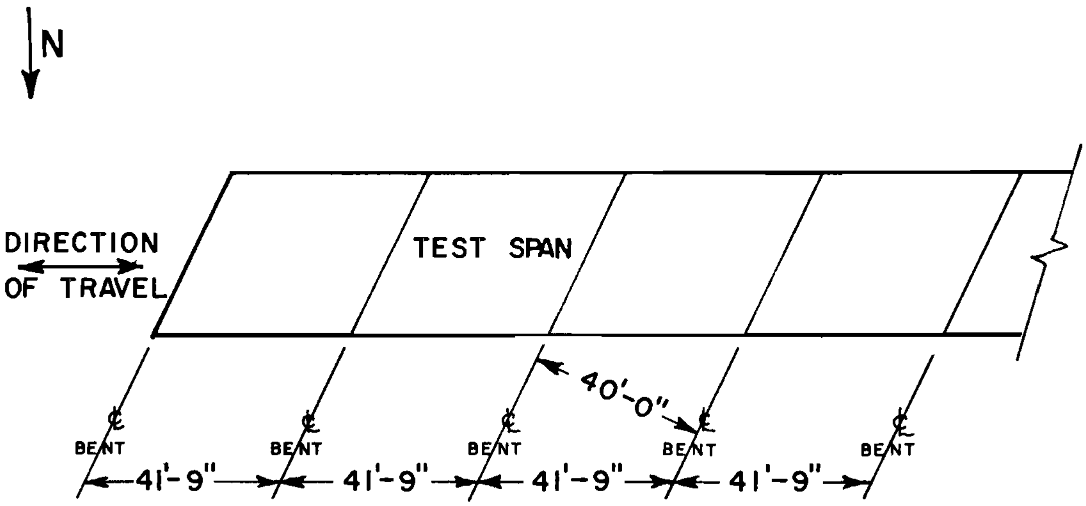
After four days the forms were removed. This was accomplished by removing the end braces and lowering the forms using the motor crane. A hole had been formed in each stem crown at midspan in which the line from the crane was inserted. This allowed the form to be lowered slowly after the supports were removed. The six dead load wires were the only obstructions. It was feared that they might be cut during form removal. The wires were made long enough so they could be pulled to one side during form removal. The dead load gages were attached to a strain indicator and read after each form was removed. The results from the dead load readings will be discussed in Chapter V.

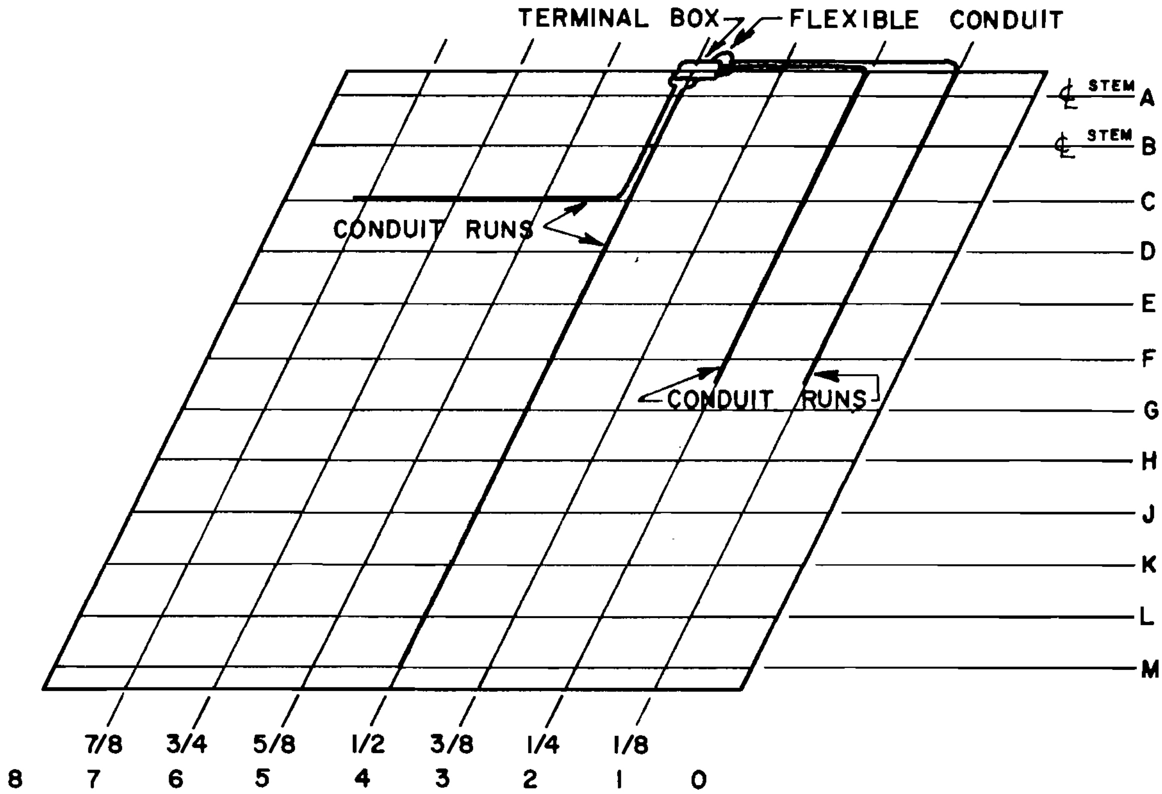
3.3 Strain Gage Hookup

With removal of the forms, the aluminum plate system with the terminal strips was exposed. The aluminum plates were easily removed and lead wires connected. These leads were run through conduit connected to the underside of the bridge. Although such wires are usually taped to the concrete or left hanging loose, in this case conduit was used to prevent the wires from being pulled out either accidentally or on purpose (by vandals). The conduit intersected at a common point and was connected to a terminal board. The conduit and terminal location is indicated in Fig. 3.2. The terminal box and conduit were watertight to ensure protection of the leads during periods of absence from the test site.

The strain measuring equipment could not be safely left on the job between reading periods, so the terminal board was used to make a convenient hookup to that equipment. A platform designed to hang from the bridge was erected to provide easy access to the terminal board and give an area to work from during testing.

Erection of the movable dial gage system was made after the conduit was in place.





TEST SPAN SHOWING GRIP SYSTEM

FIG. 3-2 CONDUIT L'AYOUT

CHAPTER IV

TESTING

4.1 Instrumentation

Since a large number of strain readings were required for the testing program, a self-balancing, direct strain reading, digital indicator was used. This instrument solved the problem of the time consuming hand-balancing operation. Each gage was connected to a switching unit so the strain of any particular gage could be displayed on the strain indicator. For convenience, each gage was connected through a variable resistor so an initial reading could be selected for each gage. The balancing unit had varying settings available to give a variety of precision to the strain read-out. An accuracy of one microinch per inch was used for this testing.

To account for expected temperature changes throughout testing, a temperature compensating gage was included in the system. A gage was applied to a bar of the same size as the bridge moment reinforcing. It was connected in the same manner and afforded the same protection as the test gages. This bar (about two feet long) was cast in a concrete beam with the same clear cover as the bridge beam. With the temperature compensating gage wired into one leg of the wheatstone bridge, no further temperature correction was necessary.

4.2 Loading

The design of the bridge span under study was based on the load applied by the AASHO H-20 design vehicle.[1,11] Unfortunately, a truck of that dimension with the required weight would be illegal on Texas highways, principally due to distribution of the rear axle weight.[11] Therefore, a truck which closely approximated the design vehicle was used. A comparison of the sizes and weights of the respective vehicles is shown in Fig. 4.1. The actual vehicle used was a ten cubic yard dump truck filled with sand until the total weight was that of the H-20 vehicle. The major difference between the vehicles was that the AASHO truck assumed a single rear axle, while the test vehicle had two closely spaced rear axles. The weight of the two rear axles was the same as the weight of the single axle of the H-20 truck.

The standard testing procedure followed was to take an initial set of zero load readings, park the truck at the appropriate location on the bridge and record the loaded readings. After readings for the particular load location were recorded, the truck was removed from the bridge and the zero readings were re-recorded, becoming the zero load readings for the next load condition. This procedure highlighted any change in the initial readings due to cracking or strain indicator drift.

The bridge was loaded at positions judged to give a variety of load conditions to match the model test program in the shortest amount of time. Three types of loading were used. The first used a single truck, the second used two trucks, while the final used three trucks. Table 4.1 lists the loading locations for the trucks. The grid system is the same as the one used for identifying strain gage locations. As

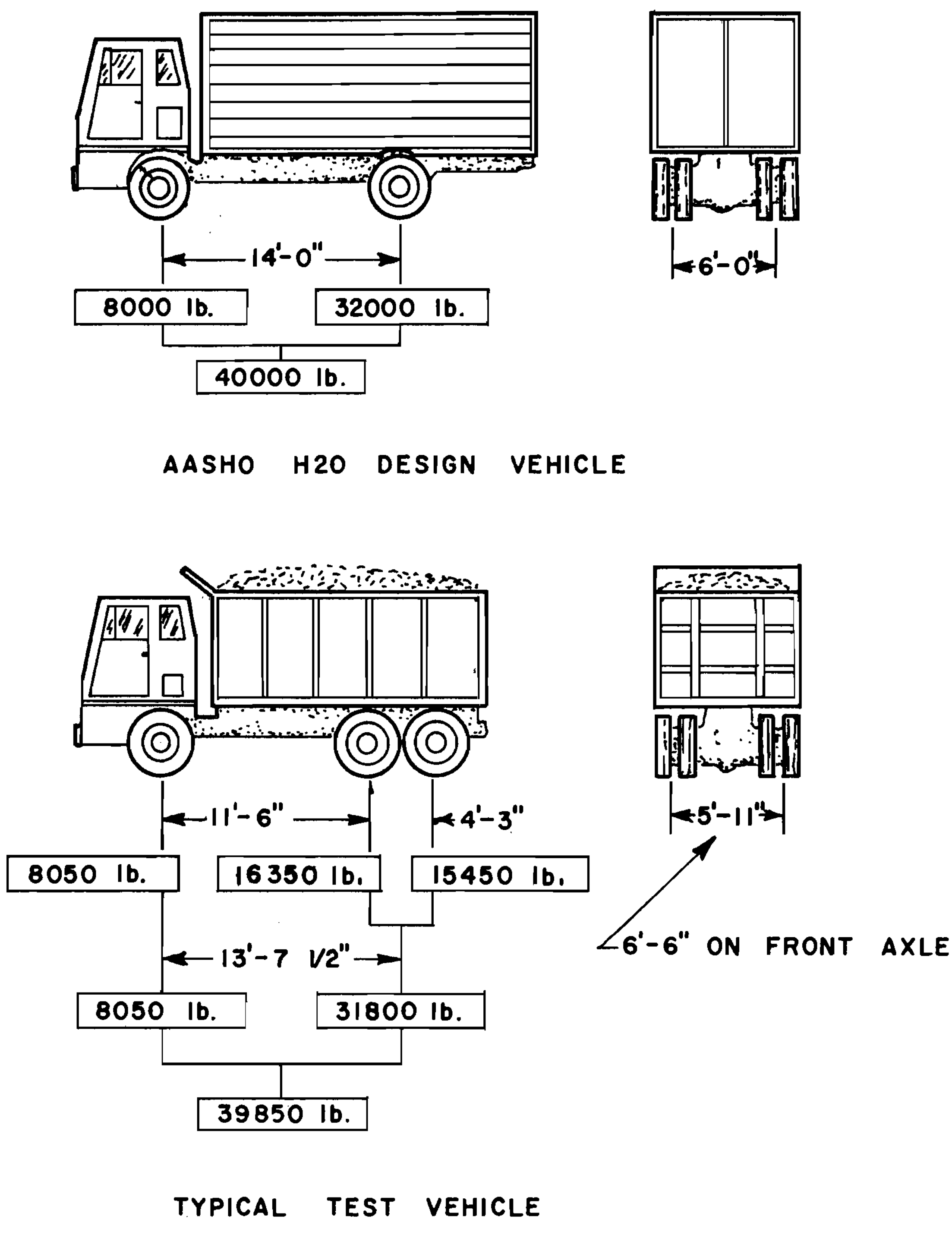


FIG. 4-1 COMPARISON OF AASHO H20 DESIGN VEHICLE AND TEST VEHICLE

TABLE 4.1. LOAD LOCATIONS

Location of Truck	Axles	Direction
B2-D2	1,2,3	W
B4-D4	1,2,3	E & W
C2-E2	1,2,3	W
C4-E4	1,2,3	E & W
E2-G2	1,2,3	W
E4-G4	1,2,3	E & W
F4-H4	2,3	W
H2-K2	1,2,3	W
H4-K4	1,2,3	E & W
J2-L2	1,2,3	W
J4-L4	1,2,3	E & W
BC4-DE4	2,3	E
HJ4-KL4	2,3	W
B4-D4; E4-G4	2	E
B4-D4; G4-J4	2	E
B4-D4; J4-L4	2	E
G4-J4; D4-F4	2	E & W
J4-L4; B4-D4	2	W
J4-L4; D4-F4	2	W
BC2-DE2; EF2-GH2; HJ2-KL2	2	E
BC4-DE4; EF4-GH4; HJ4-KL4	2	E

an example: truck B4-D4; 1, 2, 3; E & W; had the wheels of one axle over location B4 and D4, strain readings were taken with each of the three axles located over that point in turn, and the truck was pointed east and then west. Axle one is the front axle, two the middle axle, and three the rear axle. Load locations noted such as BC4-BE4 mean the truck wheels were located midway between the two girders mentioned.

The bridge was marked with chalk lines corresponding to the preset grid system. The trucks could be easily spotted at the proper location. In addition to the truck drivers a crew of five was used in testing. Two men were used for strain readings (one reader and one recorder) and three for dial readings (two readers and one recorder). The actual testing took several days spread over a period of several months. The schedule varied with availability of drivers and trucks and the need to evaluate data before subsequent loadings.

When multiple trucks were placed on the bridge, all trucks pointed in the same direction, simulating a passing situation.

The results of this testing will be presented in Chapter V.

C H A P T E R V

TEST RESULTS

5.1 Introduction

The main purpose of the full scale test program was to investigate the reliability of the laboratory tested models. The secondary purpose was to investigate the feasibility of field testing a structure of this size using improved techniques for mounting a large number of strain gages on the reinforcement. This chapter will deal primarily with the comparison of data obtained in the field with that from the laboratory tested models.

5.2 Single Truck Comparisons

Enough field data were taken to obtain fairly complete and conclusive information for single trucks with the rear axles located over the midpoints of girders. Data were obtained for trucks at locations other than midspan, but the non-midpoint loadings resulted in generally low and thus less sensitive strain readings. The data obtained from these non-midpoint loadings could not be considered significant. Also, the point of greatest interest to the designer is that of greatest strain (or moment).

In a like manner, it was found unnecessary to consider data obtained from midspan axle placement other than the two rear axles.

The strains measured with the front axle placed at midspan are not significant when compared to strains from rear axle placement. The two rear axles being closely spaced cause the data obtained to be virtually the same with either axle over midspan. Since the middle axle placement indicated slightly higher strains, all strain plots will be for axle 2, unless noted.

To give an indication of strain magnitude for loads placed other than at midpoint, Figs. 5.1 and 5.2 are plots of the strains measured for the same truck placed with axle 2 at the span midpoint and then at the quarter point. The plots are for only one-half the bridge width, since the strains at these low levels diminish very quickly at girders further from the load point. However, these plots indicate the relationship between strains for the indicated load locations.

Figures 5.3 and 5.4 show plots of midpoint strains for subsequent placement of each of the three axles at midspan. These plots indicate the reduced significance of the strains with axle 1 at midspan and the closeness of the strains for axles 2 and 3 placed at midspan.

Figures 5.5 through 5.9 compare the measured prototype strains for axle 2, placed at midspan, with the corresponding measured model strains found by Leyendecker.[6,8] Three comparisons are presented in two forms. The lower portion at each figure compares absolute strains measured at the midpoint of each girder, while the upper portion compares the percentages of the total measured midspan strain. These comparisons indicate very similar patterns of strain distribution in both model and prototype. The qualitative agreement seems well within the accuracy expected. Later in this chapter a quantitative measure of correlation will be given for the data.

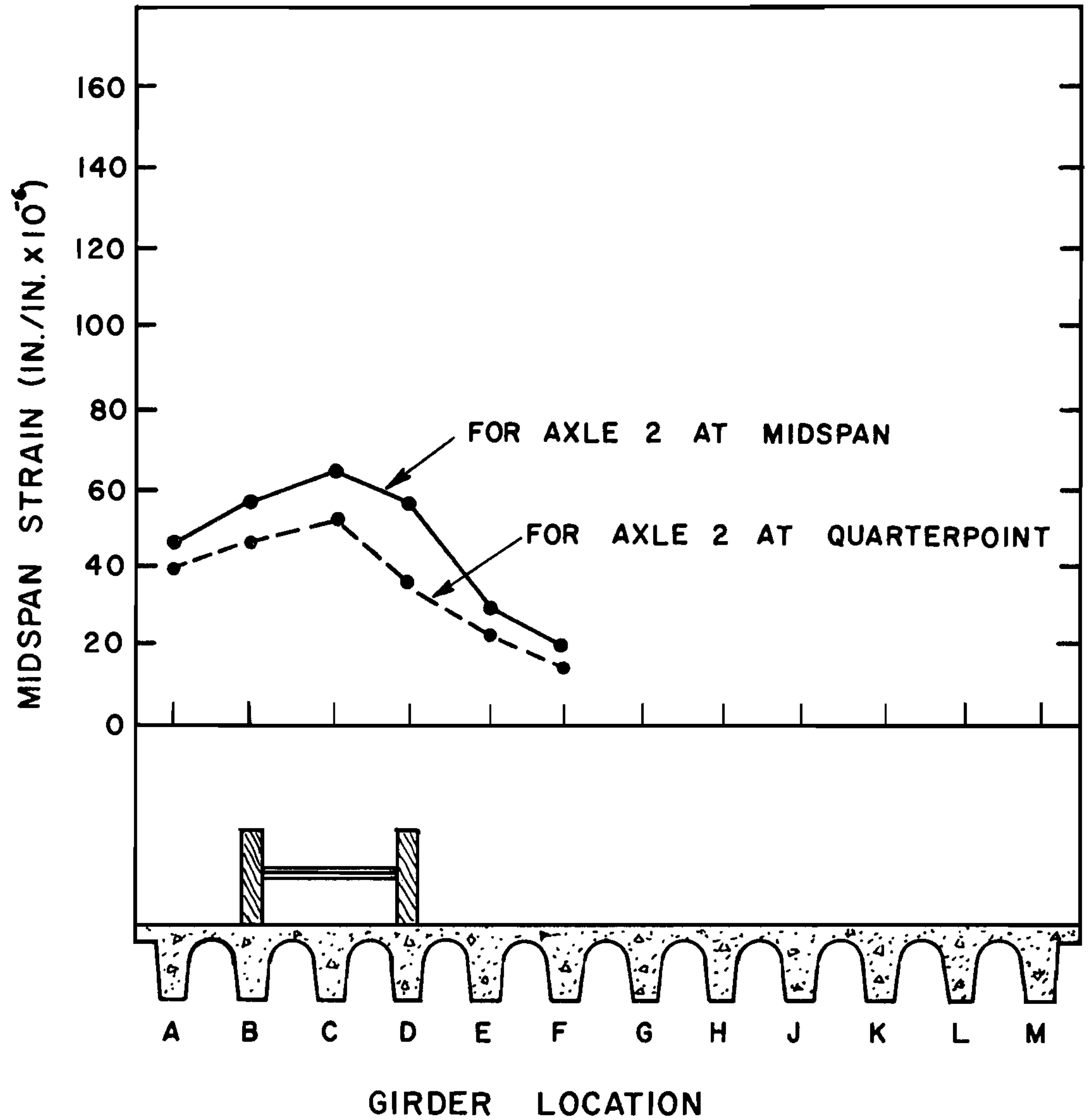


FIG. 5-1 STRAIN DATA FOR SINGLE TRUCK AT MIDSPAN AND QUARTERPOINT, B4-D4

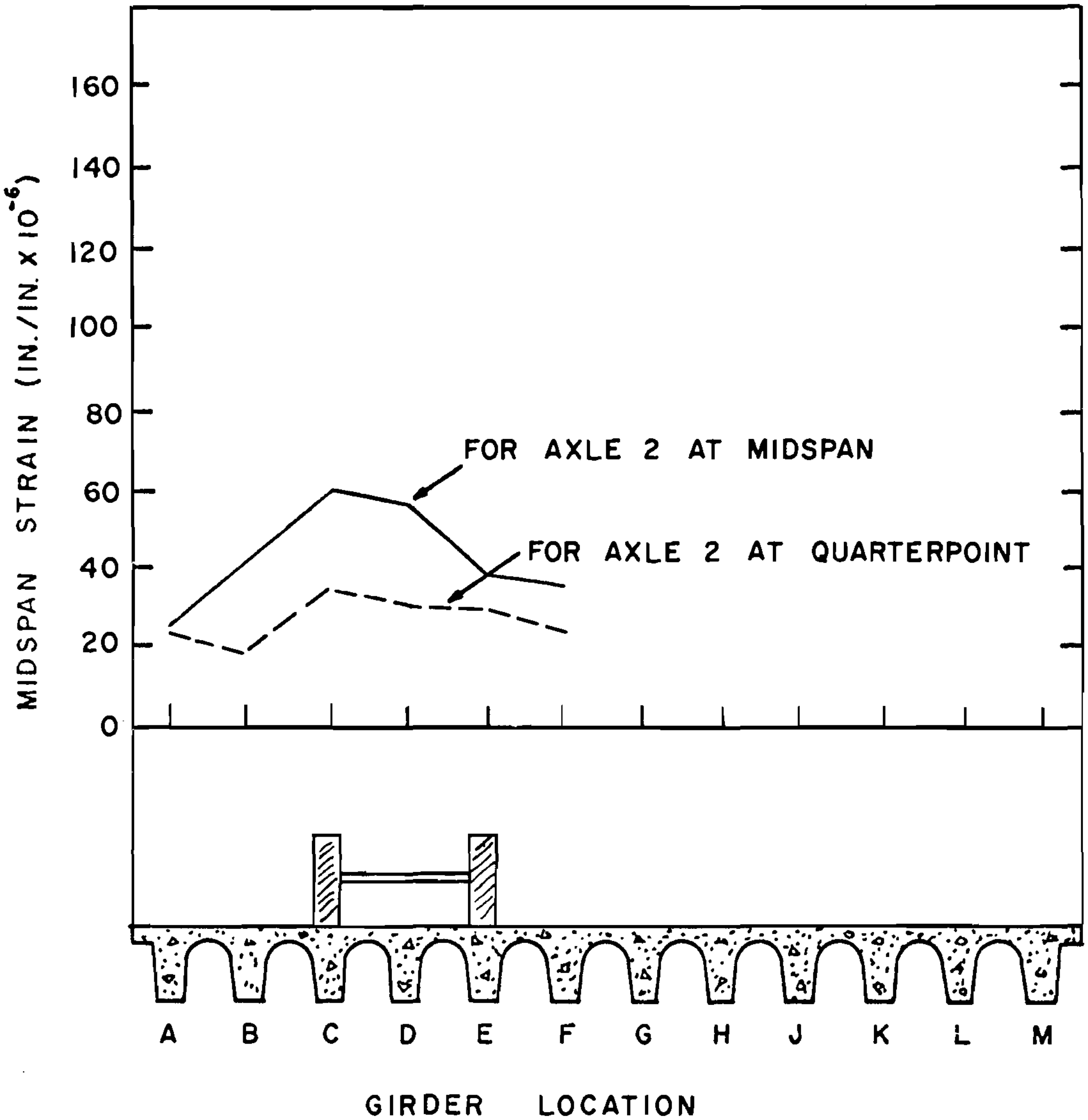


FIG. 5-2 STRAIN DATA FOR SINGLE TRUCK AT MIDSPAN AND QUARTERPOINT, C4-E4

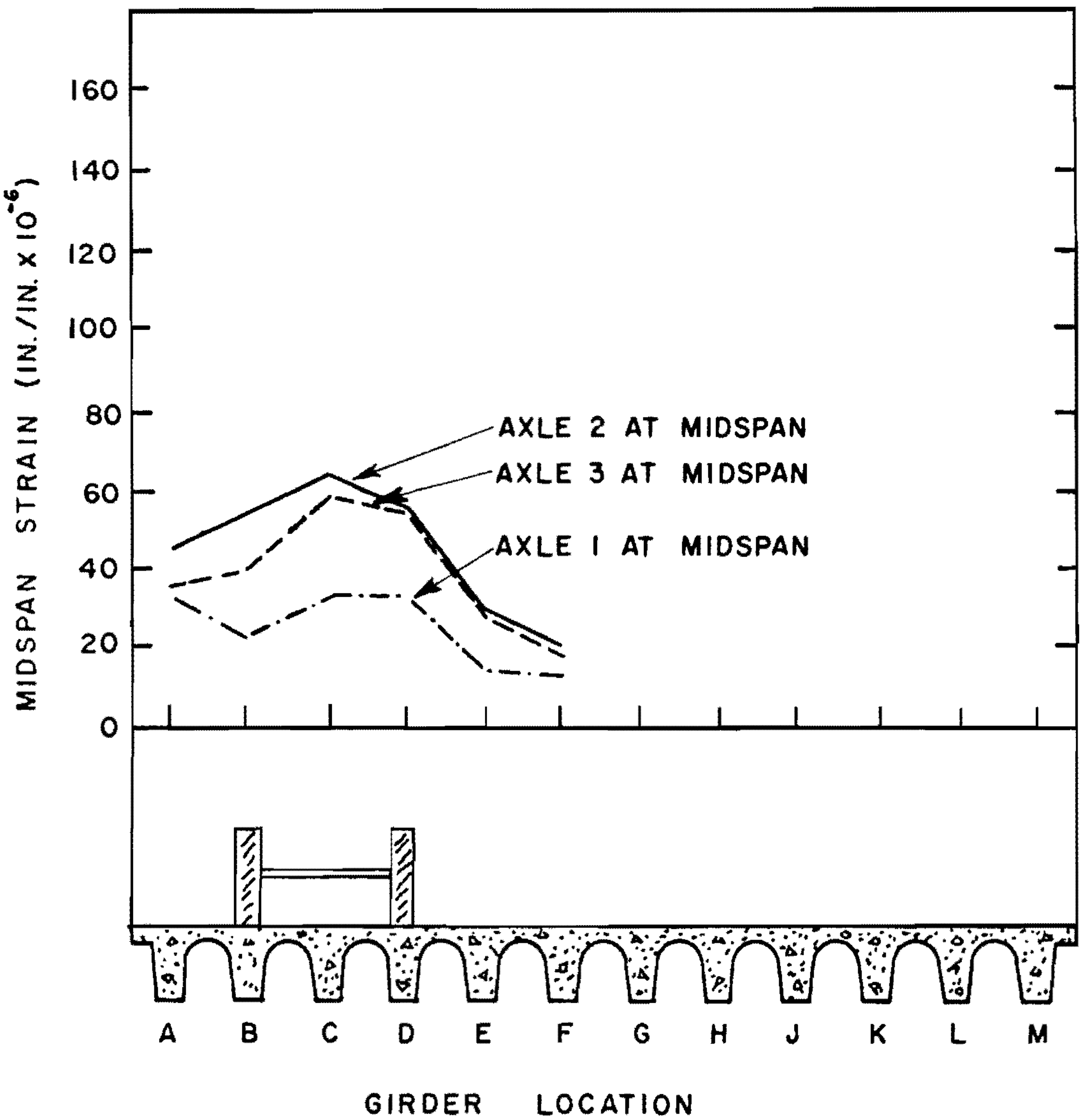


FIG. 5-3 STRAIN DATA FOR SINGLE TRUCK AT MIDSPAN, B4-D4

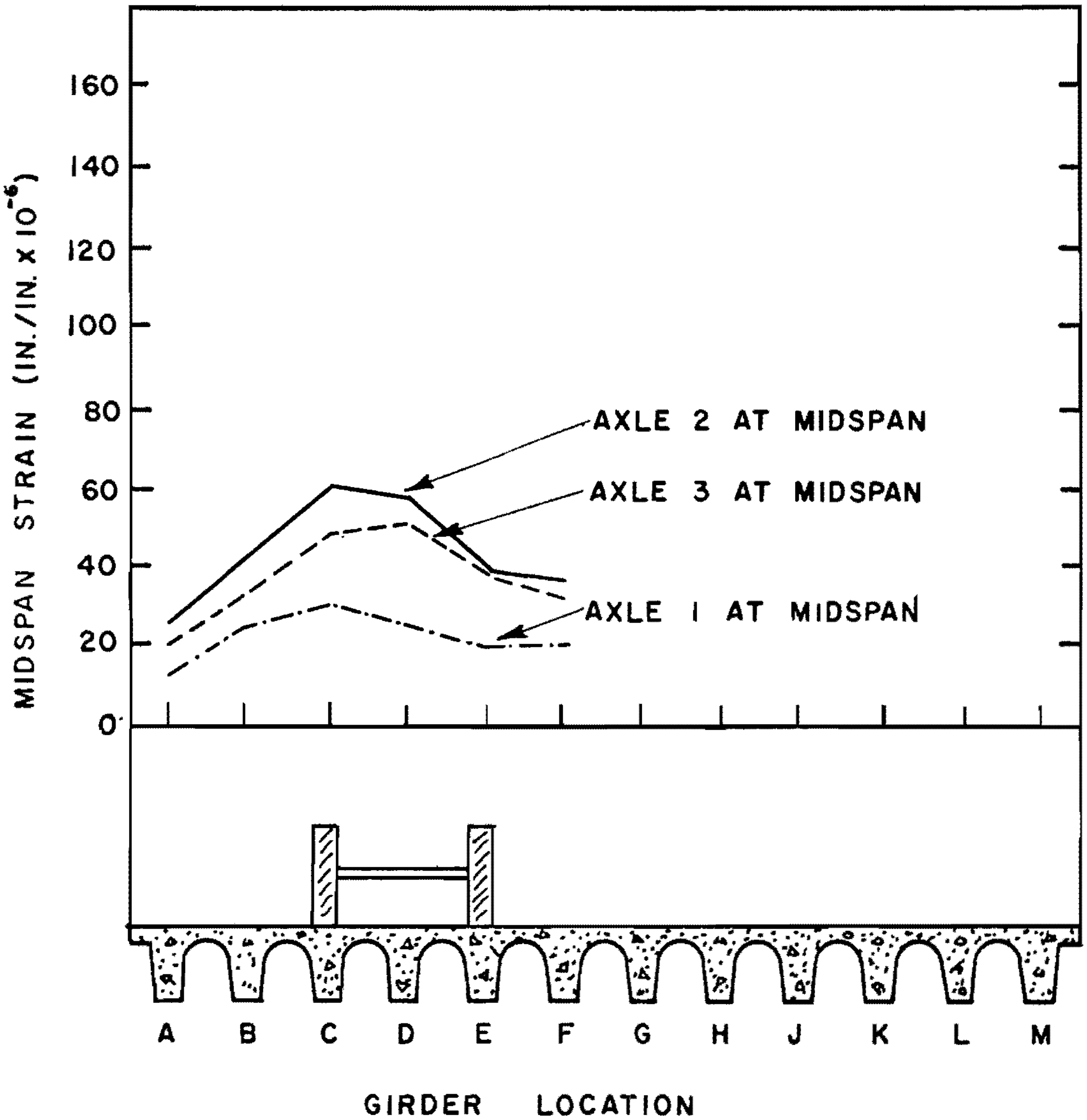


FIG. 5-4 DATA FOR SINGLE TRUCK AT MIDSPAN, C4-E4

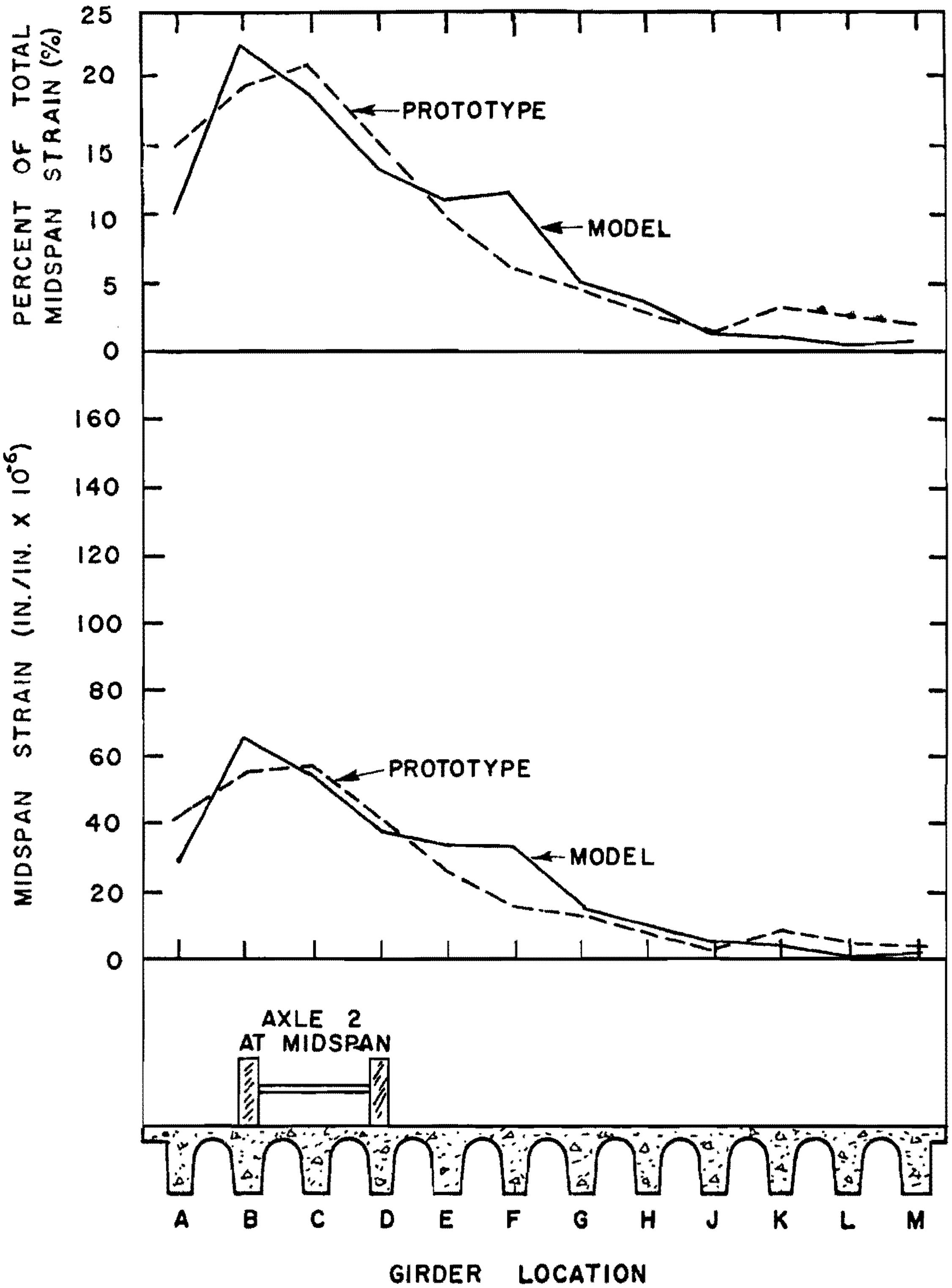


FIG. 5-5 STRAIN DATA FOR A SINGLE TRUCK, B4-D4

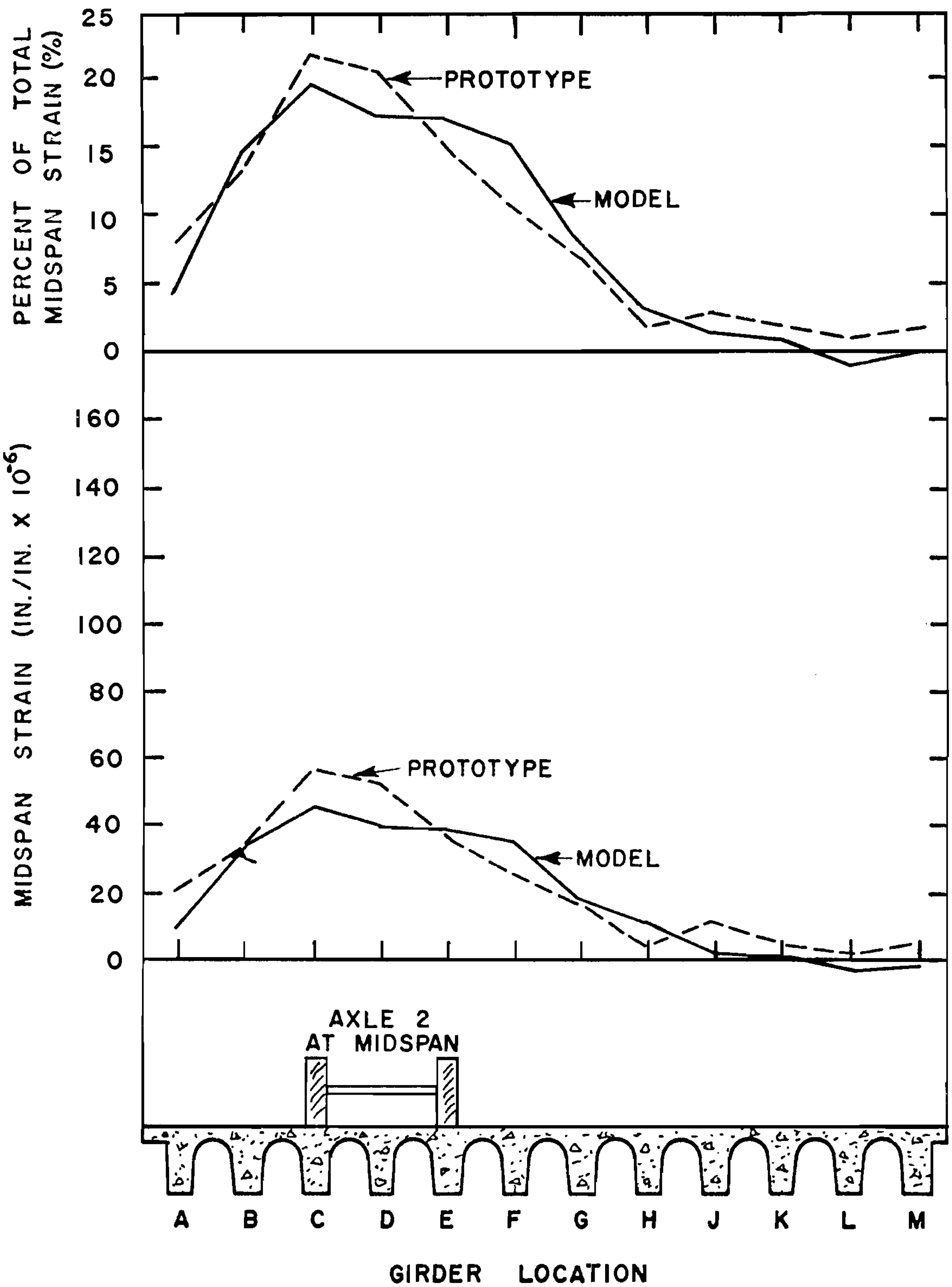


FIG. 5-6 STRAIN DATA FOR A SINGLE TRUCK, C4-E4

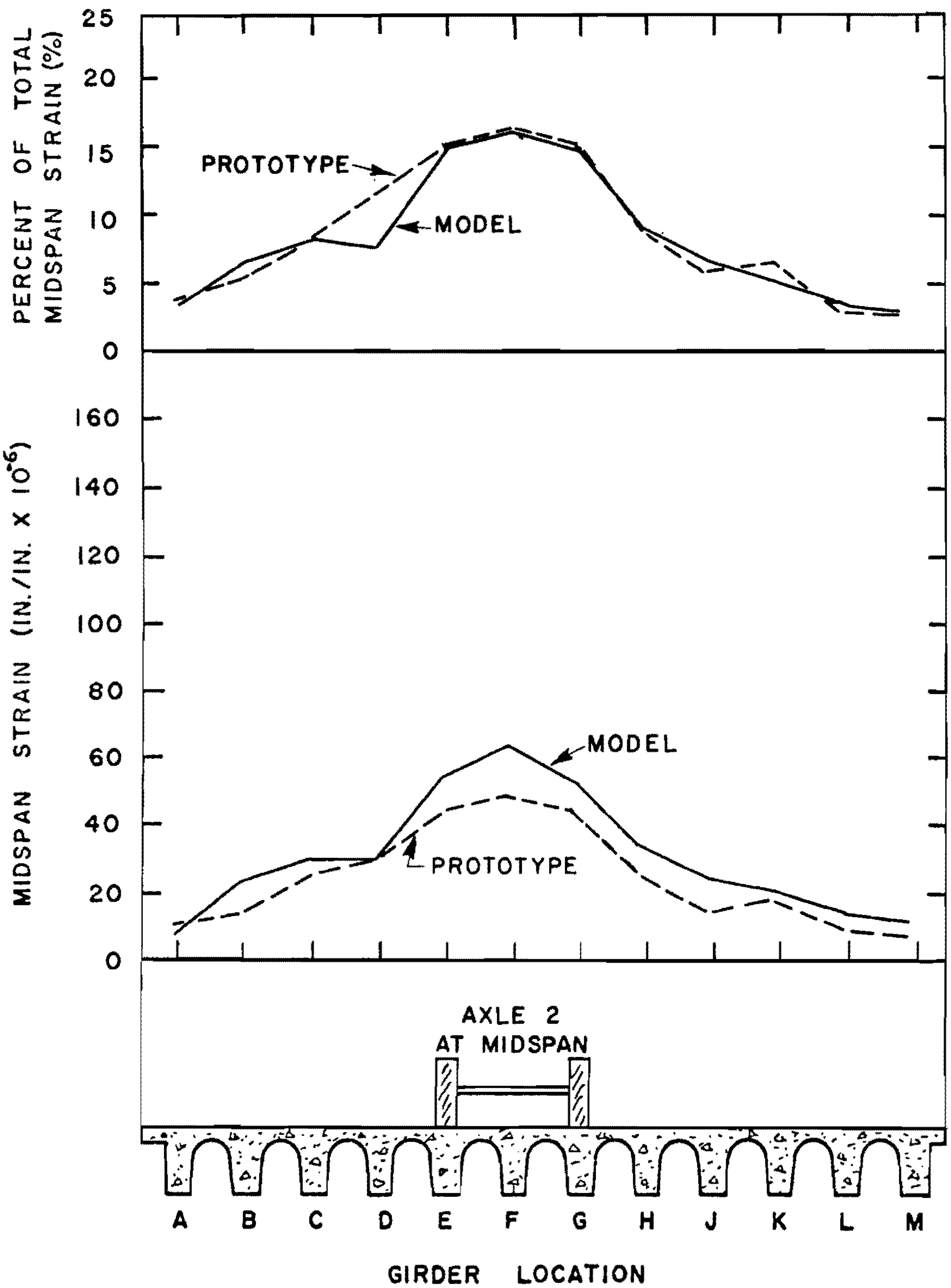


FIG. 5-7 STRAIN DATA FOR A SINGLE TRUCK, E4-G4

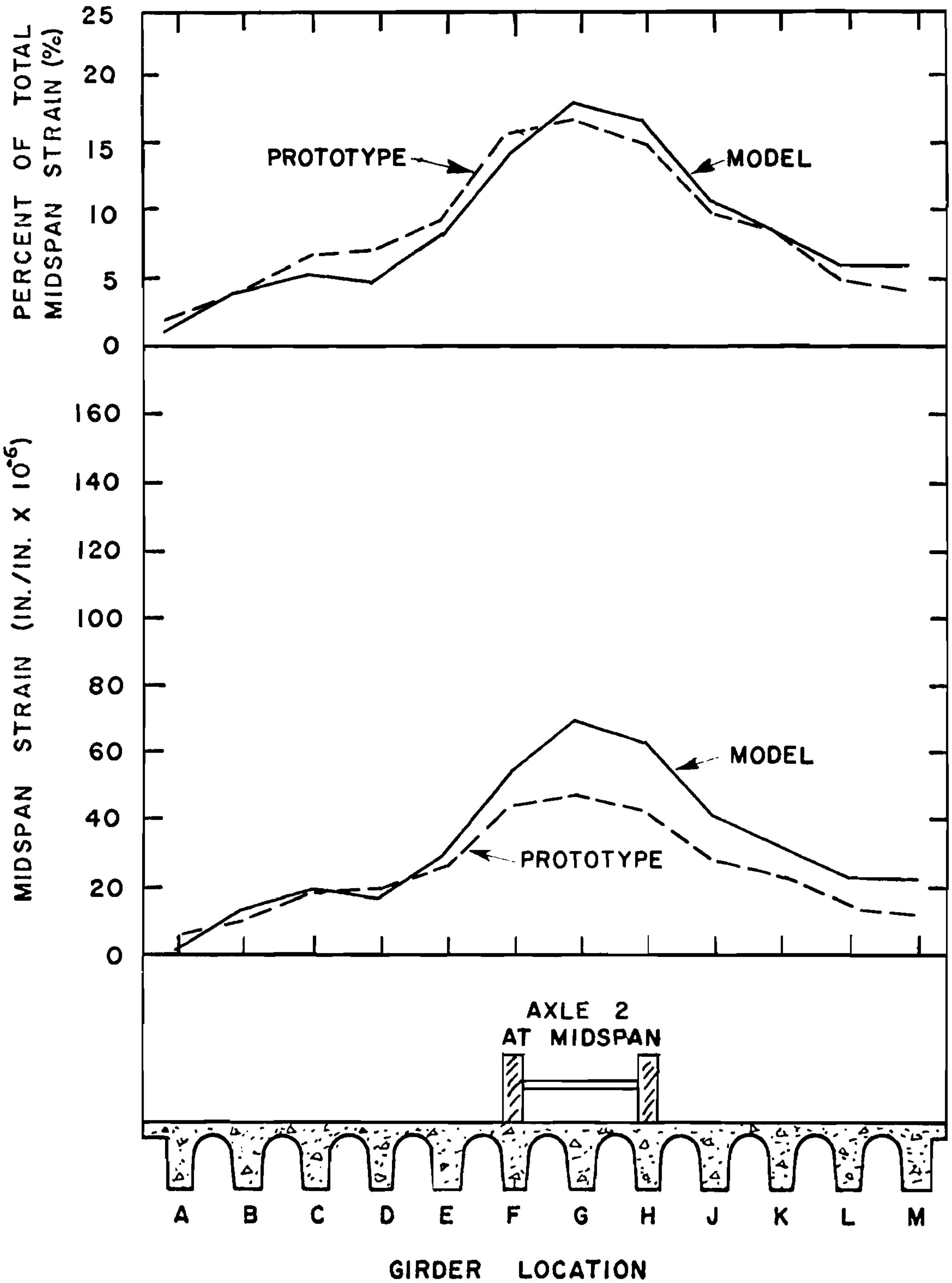


FIG. 5-8 STRAIN DATA FOR A SINGLE TRUCK, F4-H4

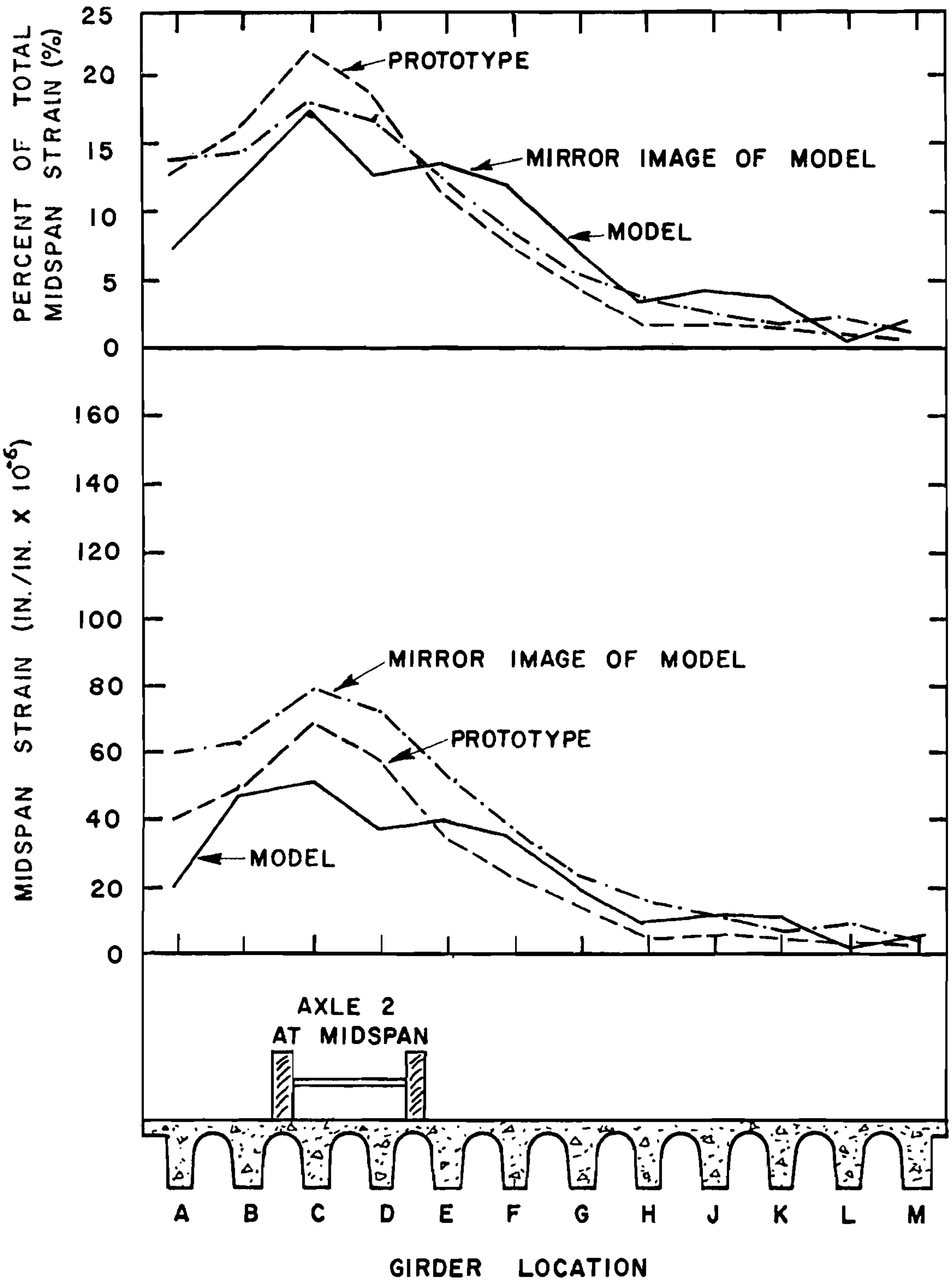


FIG. 5-9 STRAIN DATA FOR A SINGLE TRUCK, BC4-DE4

In Fig. 5.9 an extra set of data is included. Additional model data were available for the model truck at location HJ4-KL4 with the truck pointed in the opposite direction. The data for this mirror image of truck BC4-DE4 are included. The model and prototype strains do not show a very high degree of correlation, principally because of the discrepancies of strain readings at gage stations C and D. The discrepancies are reflected in both the absolute strains and percentage strains. By reversing the plot (plotting point A as M, B as L, etc.) it gives a measure of strain at location BC4-DE4. The shape of the curve and the magnitude of the strains agree much more closely with the prototype data.

Generally the model strains are somewhat higher than the prototype strains. This can be explained by the fact that due to the testing schedule the model had been loaded extensively at service load levels, while the prototype had not been previously loaded, since it was not yet open to traffic. Therefore, it can be assumed reasonably that the model had undergone more local cracking than the prototype. Comparison of other data substantiates this theory. Therefore, considering this cracking, very good agreement was obtained between model and prototype strains.

5.3 Double Truck Comparisons

Four sets of data were obtained for pairs of trucks located at different points on the span. In each case the trucks were pointed in the same direction and were stationed at various girder locations, as given in Table 4.1. Results for these double-truck locations are shown in Figs. 5.10 through 5.13. In two instances (see Figs. 5.10 and 5.11) both absolute and percentage strain agree very well. In the other two cases, the midspan strain plots indicate relatively poor agreement. In

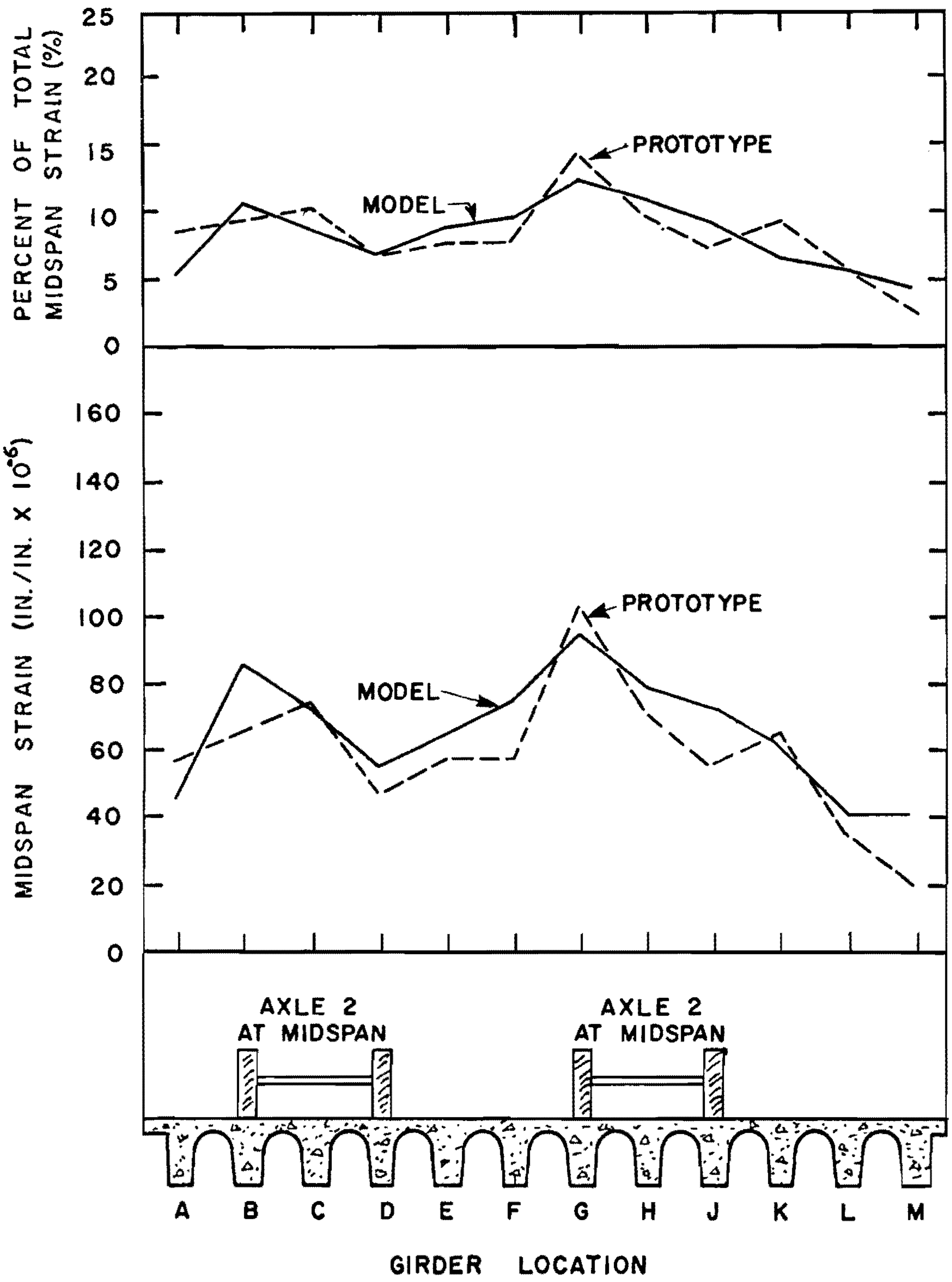


FIG. 5-10 STRAIN DATA FOR DOUBLE TRUCKS, B4-D4 AND G4-J4

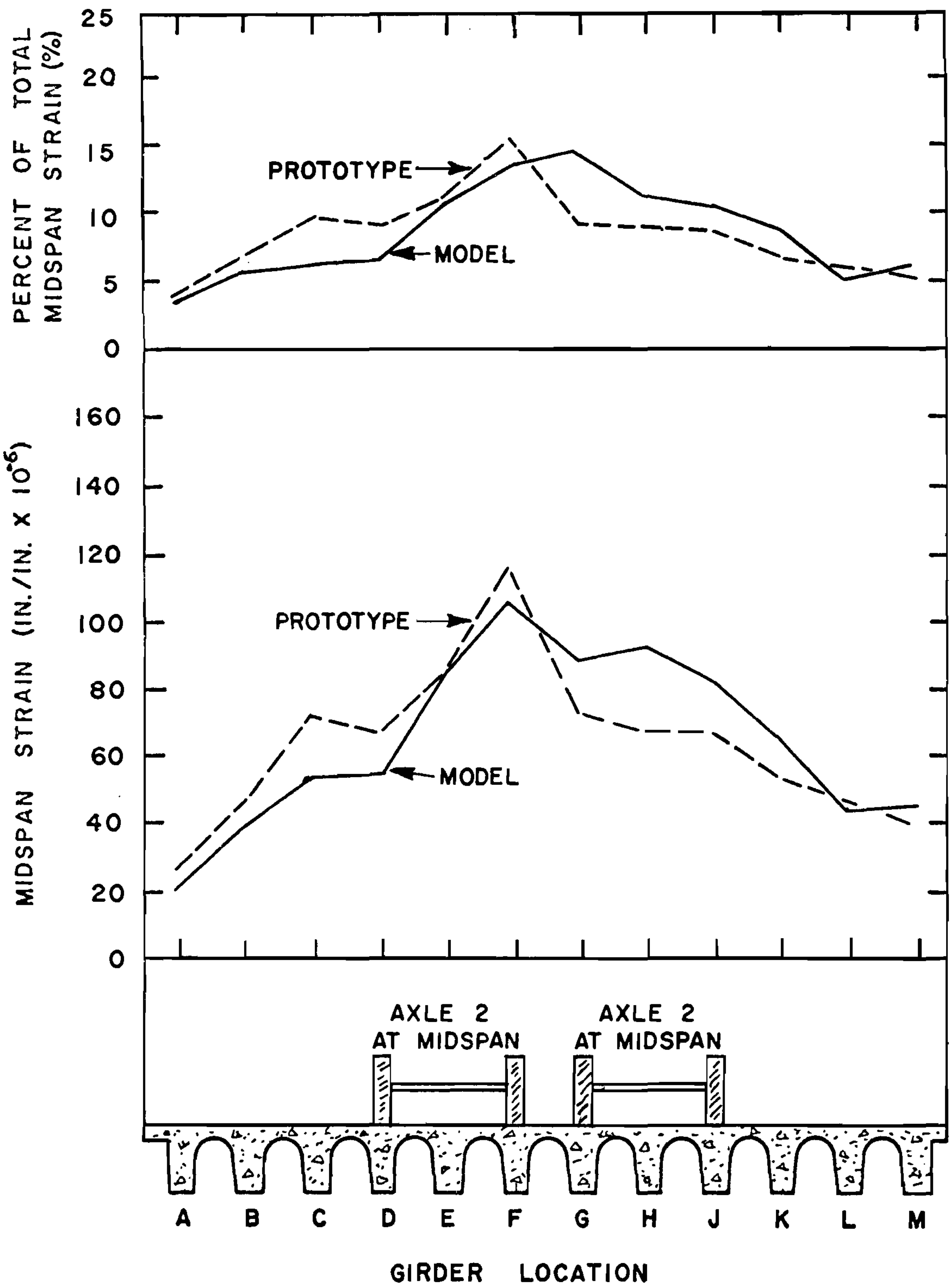


FIG. 5-11 STRAIN DATA FOR DOUBLE TRUCKS, D4-F4 AND G4-J4

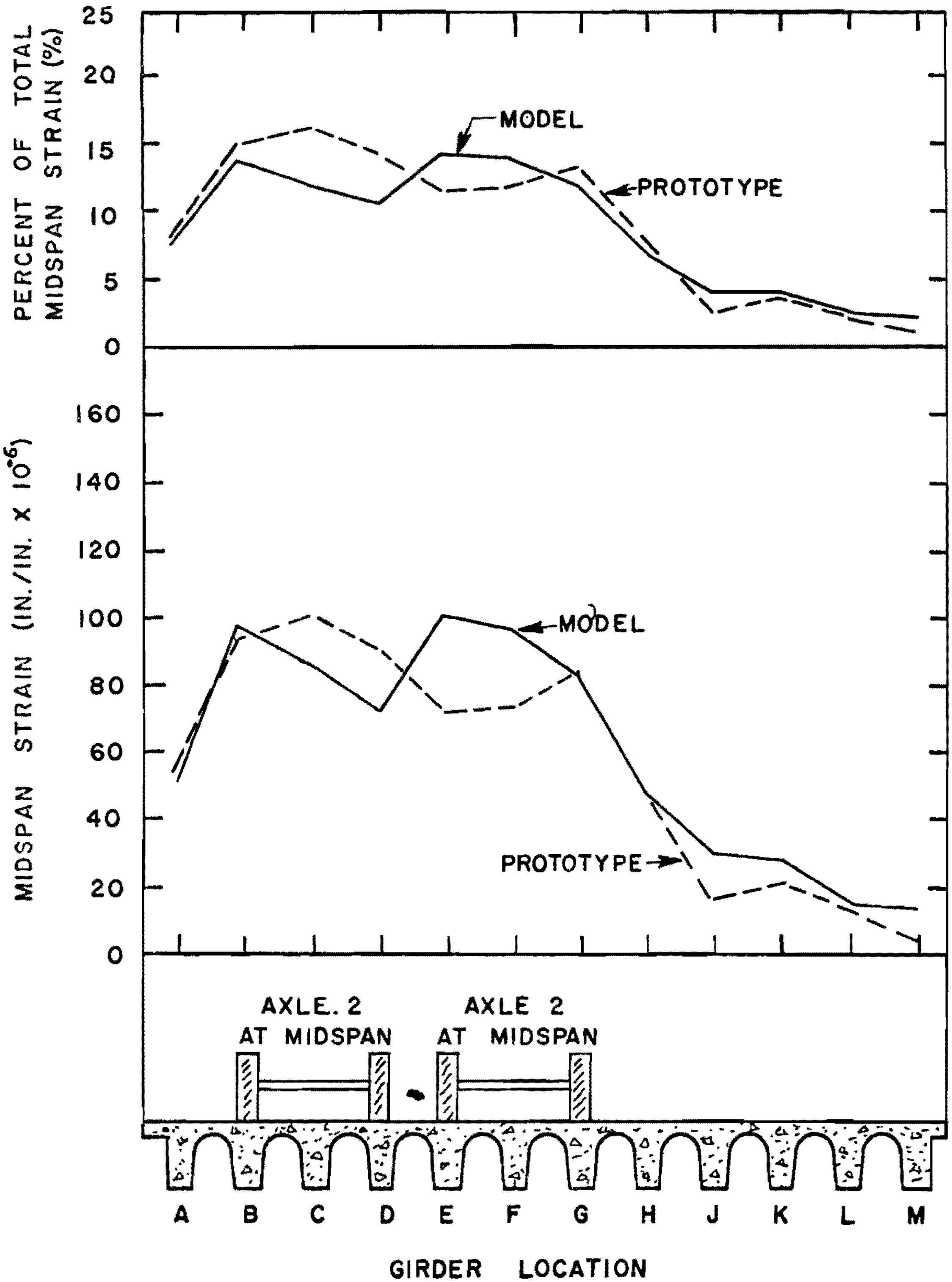


FIG. 5-12 STRAIN DATA FOR DOUBLE TRUCKS, B4-D4 AND E4-G4

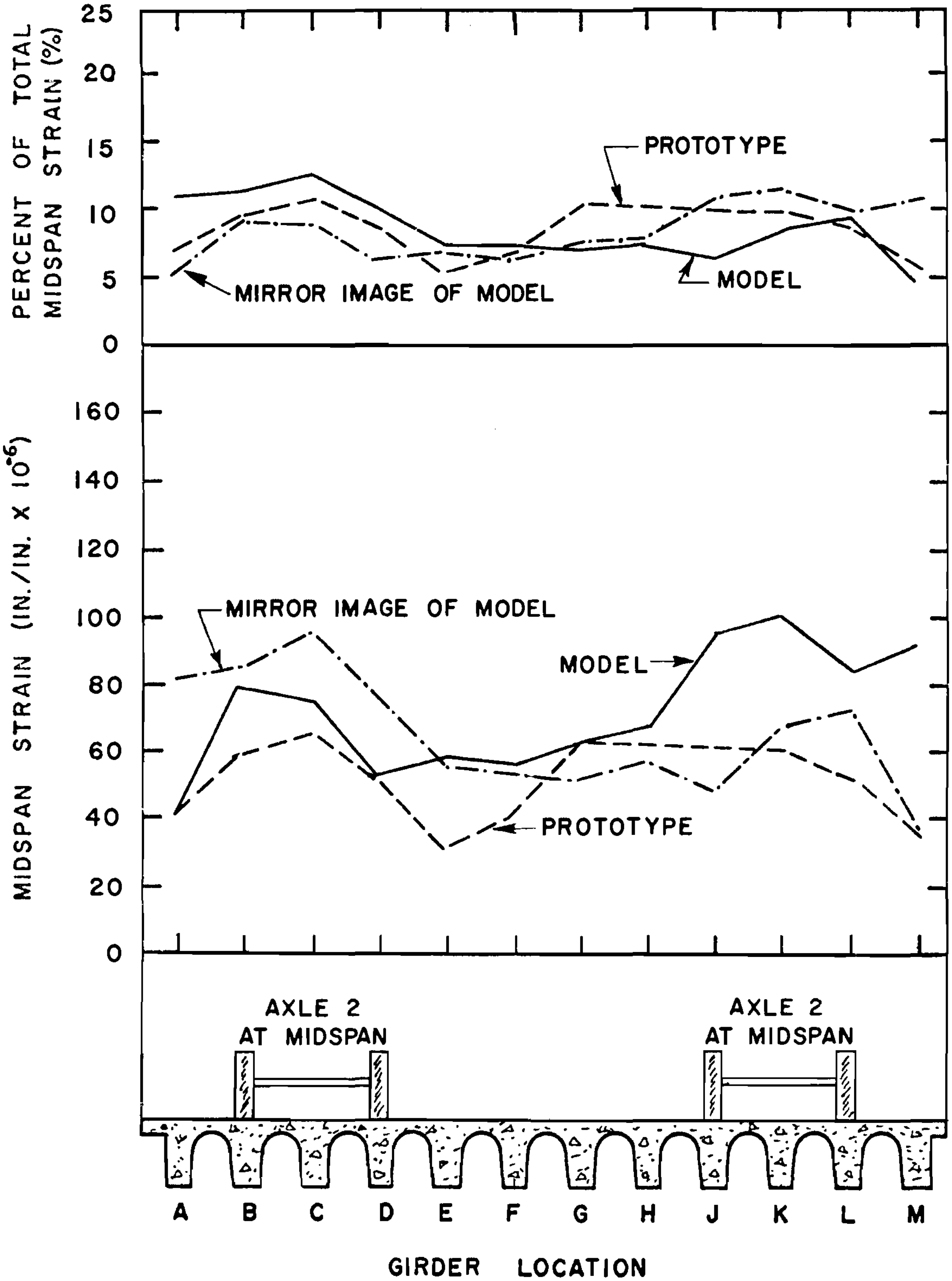


FIG. 5-13 STRAIN DATA FOR DOUBLE TRUCKS, B4-D4 AND J4-L4

the case of trucks at B4-D4 and J4-L4 (Fig. 5.13) the model truck at J4-L4 shows a higher strain than the corresponding prototype truck or either truck at B4-D4. This again indicates the condition of a higher state of cracking in the model in the region of girders G through M. The data from the model mirror image truck were plotted and the plot indicates a little better correlation with prototype values for girders G through M. Also, on the prototype, the gage at location M4 (location E4 for the mirror image truck) gave an extremely erratic reading. For the purpose of plotting percent strain, the point was interpolated from the strains of the two adjacent girders.

5.4 Triple Truck Comparisons

Only one test was run for three trucks simultaneously on the bridge at midspan, and the results are shown in Fig. 5.14. The results of the absolute strain plot do not show good agreement, but the percent plot shows better correlation. Again, the lack of data agreement is probably due to the greater previous cracking history of model girders G through M.

5.5 Correlation[10]

Definite conclusions cannot be drawn from the data as presented in Figs. 5.5 through 5.14. Visual inspection and the tester's judgment can form an overall impression of the correlation. An expression of correlation of data is available by plotting measured absolute midspan strain of prototype model for each load location. The plot is shown in Fig. 5.15. A perfect fit would be represented by the 45^{o} straight line labeled "ideal correlation." A less-than-perfect fit is shown for the

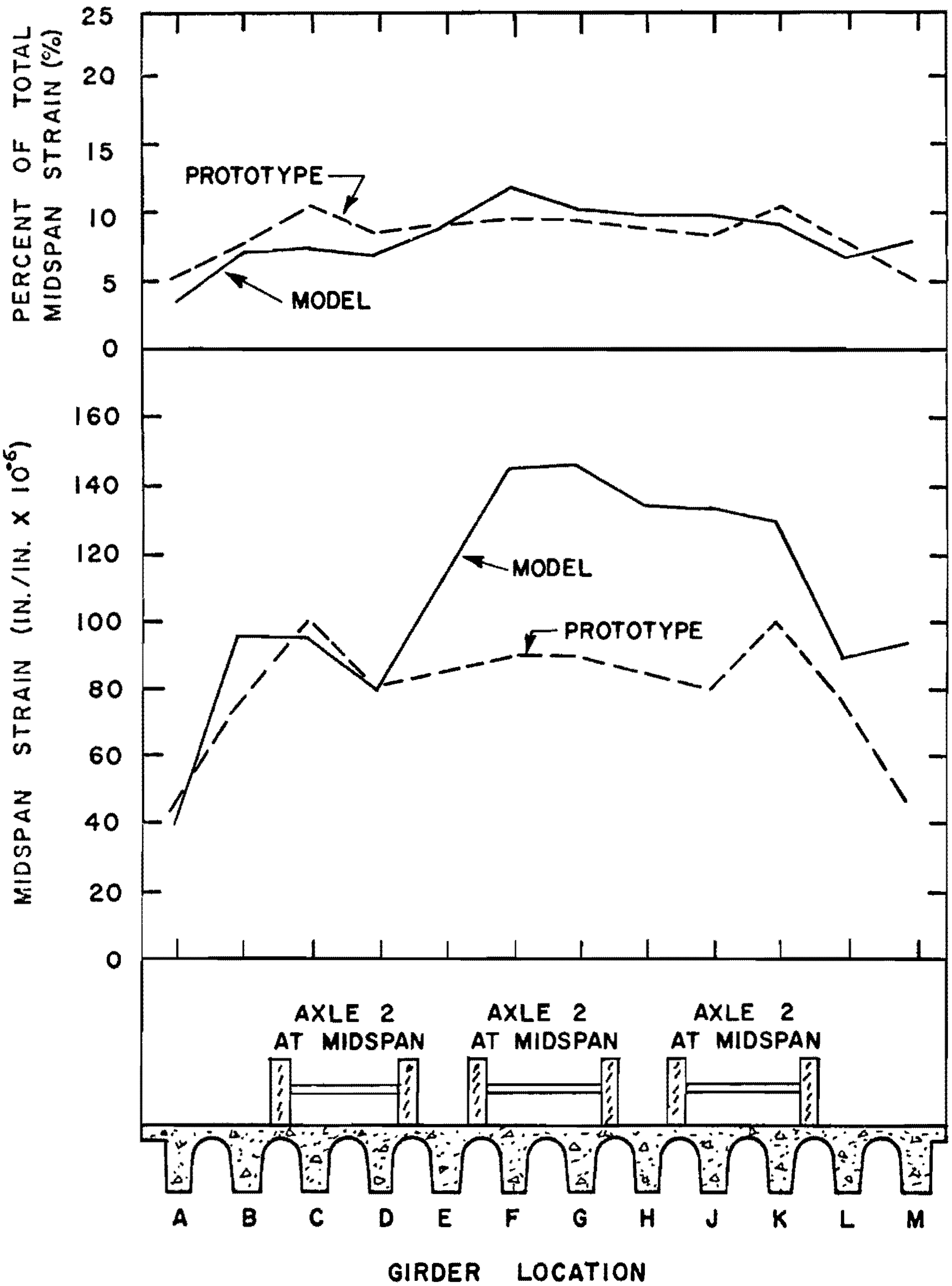


FIG. 5-14 STRAIN DATA FOR TRIPLE TRUCKS

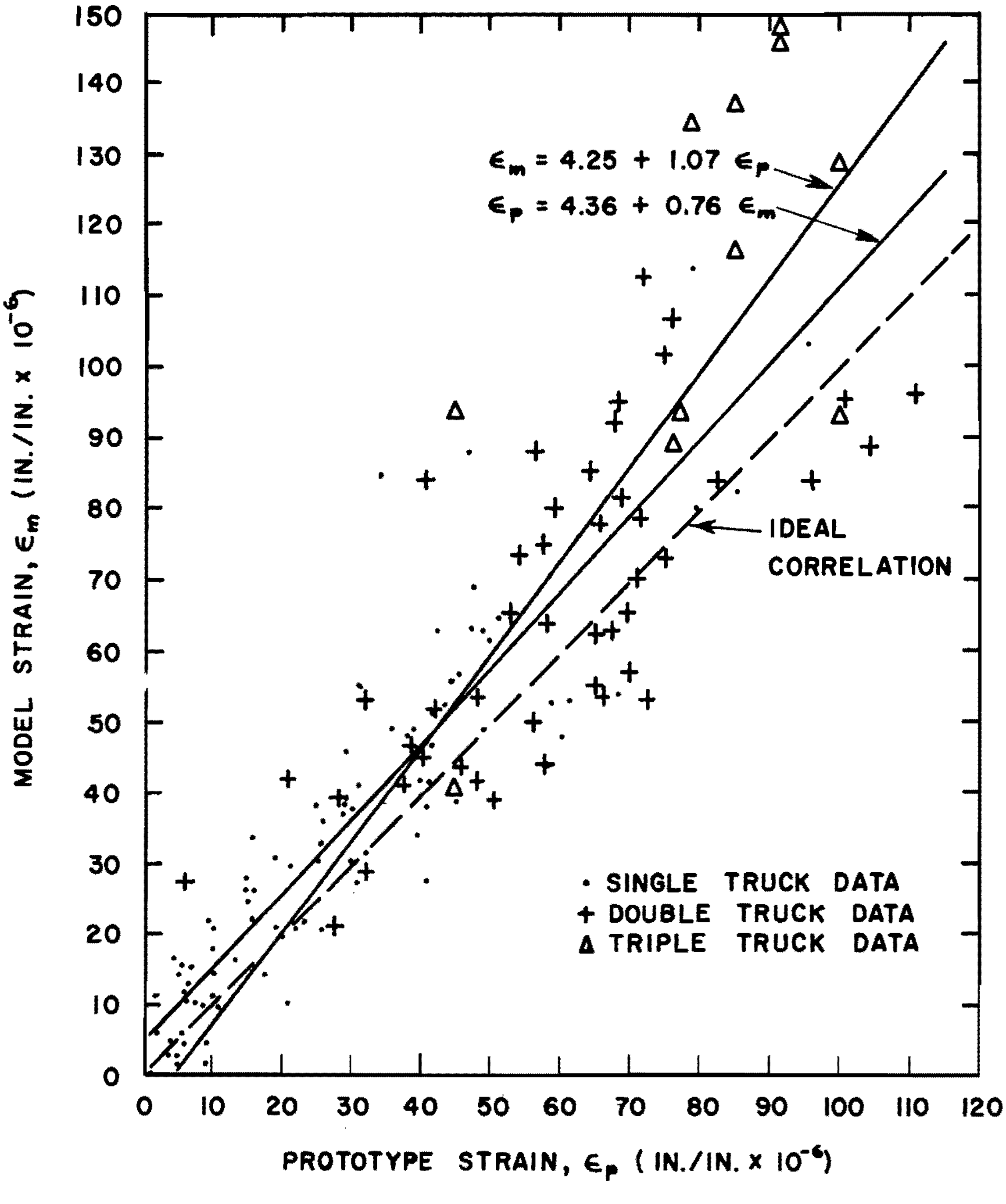


FIG. 5-15 SUMMARY OF MODEL AND PROTOTYPE DATA WITH REGRESSION ANALYSIS

model as the dependent variable and again with the prototype as the dependent variable. The equations for the lines were found by using a least squares fit of the data. Using either of the equations, a coefficient of correlation is found as +0.90. The range of the coefficient of correlation is from -1.0 for negative correlation to 0.0 for no correlation to +1.0 for perfect correlation. A coefficient of +0.90 indicates very good correlation.

5.6 Dead Load Strain and Deflections

Before actual service live load data were taken, dead load readings were made. These readings were taken immediately after removal of form work and at certain unspecified intervals afterward until load testing began. These readings behaved erratically at each position monitored. Due to the length of time that elapsed between the zeroing of the gages, placing of concrete, removal of forms and actual testing, the strain values recorded drifted considerably, often in opposite directions. The data are refined as much as possible to give an indication of dead load strain. Figure 5.16 shows a time versus strain plot for the data at gage location F4, which is the only gage that indicated reliable readings. For days 1 through 14, the strain is constant at approximately 200 microinches per inch. (Approximately 6000 psi steel stress.) Thus, the dead load stresses represent a more significant portion of total stress than did any service live load measurements. After the fourteenth day, the recorded strain increased rapidly. A possible reason is at that time all the spans for this bridge were completed and construction equipment was allowed to move across the span, causing some cracking of the concrete in the region of the gage. This assumption is partially conjectural, since no "qualified"

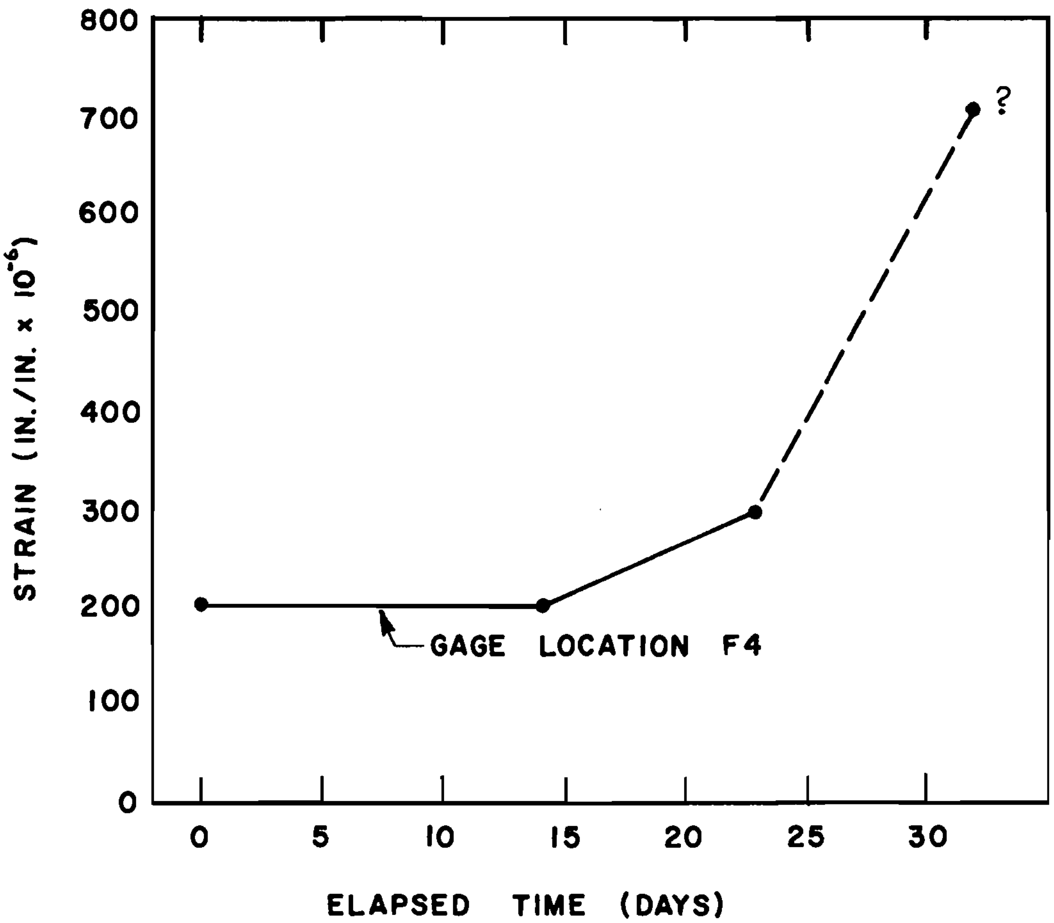


FIG. 5-16 DEAD LOAD STRAIN DATA

observer was present to observe exactly the conditions, although some cracking was visible to the naked eye. The readings may simply reflect the inaccuracy of procedures used for time dependent readings. Since the recording was during the fall season, the wide range of temperature between day and night could adversely affect the gages or recording equipment. The continual connecting and disconnecting of equipment between test days could have caused additional errors. These factors did not influence live load readings.

For each midpoint loading case deflection readings were taken using the deflection bridge assembly. Due to high winds at the test site, which caused motion in the deflection bridge, the readings were unreliable and must be classed as worthless. The deflections were taken as a check on strain readings in case such a check was needed. Fortunately, these readings were not needed.

5.7 Transverse Loading

The magnitude of strain measured by the transverse moment gages was insignificant. Table 5.1 shows measured transverse strain and calculated stress for a single truck at location B4-D4 and E4-G4.

Since the gages are close to the neutral axis of the section, the reading could be either positive or negative, indicating tension or compression. The greatest measured strain for a single truck loading indicates a figure too small to be significant. The strains measured when resolved to stress indicate values smaller than the tensile strength of the concrete. Since no cracking was apparent at that load level, this assumption must be assumed correct. Since such small transverse strains were recorded for single truck loads, the readings were abandoned for double truck and triple truck loadings.

TABLE 5.1. TRANSVERSE STRAINS AND STRESSES

Gage Location	Truck at B4-D4 (Axle 2)		Truck at E4-G4 (Axle 2)	
	Strain in./in. x 10^{-6}	Stress psi	Strain in./in. x 10^{-6}	Stress psi
AB4T*	15.0	435.0	1.0	29.0
B$T	3.0	87.0	9.0	261.0
BC4T	5.0	145.0	7.0	203.0
E4T	1.0	29.0	3.0	87.0
EF4T	1.0	29.0	7.0	203.0
F4T	-4.0	-116.0	12.0	348.0
E2T	2.0	58.0	6.0	174.0
EF2T	-3.0	-87.0	7.0	203.0
F2T	3.0	87.0	7.0	203.0

*Indicates a gage in the crown between girders A and B at location 4 (midspan) transverse to the direction of the girders.

5.8 Comparison with AASHO Specifications

The AASHO load distribution factor, K_A, which specifies the fraction of wheel load carried by a girder is expressed as

$$K_A = S/C \tag{5.1}$$

where K_A = number of wheel loads carried by a girder

S = girder spacing in feet

C = constant

This constant is dependent upon the type of system and the number of traffic lanes. For a concrete slab and girder system as discussed here with one lane of traffic, the constant is 6.0; for two or three lanes the constant is 5.0. The factor K_A determines the design moment, M_D, by using the expression:

$$M_D = K_A M \tag{5.2}$$

where M is the maximum longitudinal moment due to one longitudinal line of wheels.

The relationship between the constant and the Guyon-Massonnet longitudinal moment distribution factor, K_{GM},[9] can be expressed as:

$$C = \frac{S}{K_{GM}} \cdot \frac{N_G}{N_W} \tag{5.3}$$

where K_{GM} = ratio of longitudinal moment carried by a specific girder to the average longitudinal moment in all girders

N_G = number of girders

N_W = number of longitudinal lines of wheels

A relationship between the AASHO "C" and the experimental "C" can be found by using Eq. 5.3. Since all girder dimensions are nearly the same, a girder strain distribution factor can be assumed equal to the longitudinal moment distribution factor K_{GM}. The girder strain distribution factor is expressed as the strain of any girder divided by the average strain for all girders. For this comparison, three single truck locations and two double truck locations will be investigated. The strain distribution factors determined will be shown for the three girders immediately under the load points.

For the single truck loadings, the cases chosen are for trucks at B4-D4, C4-E4, and E4-G4 (Figs. 5.5, 5.6, and 5.7, respectively). Table 5.2 gives the results.

The factor K_{GM} was determined from the measured prototype data. The prototype constant, C, was then calculated by substituting the K_{GM} found into Eq. 7.3 using a girder spacing of three feet and a longitudinal

TABLE 5.2. SERVICE LOAD SINGLE AASHO TRUCK DESIGN CRITERIA

Load Location	Girder	K_{GM}	Prototype C	AASHO C
B4-D4	B	2.55	7.1	6.0
	C	2.74	6.6	6.0
	D	1.94	9.3	6.0
C4-E4	C	2.53	7.1	6.0
	D	2.39	7.6	6.0
	E	1.74	10.2	6.0
E4-G4	E	1.50	12.0	6.0
	F	1.64	11.0	6.0
	G	1.50	12.0	6.0

wheel line number of two. The value determined for the prototype C is much greater than that recommended by AASHO, indicating a more uniform stress distribution. As expected, the value of C gets larger as the distance from the exterior girder increases. The AASHO specification appears to be from 10 percent to 100 percent conservative for this case.

Similar checks were run for the two truck loading cases B4-D4, J4-L4, and B4-D4, E4-G4 (see Figs. 5.10 and 5.11). Table 5.3 shows the result of these computations.

The prototype C is calculated using the equation

$$C = \frac{3}{K_{GM}} \cdot \frac{12}{4} = \frac{9.0}{K_{GM}} .$$

In a few cases the prototype test determined value of C approaches the recommended AASHO "C" value of 5.0, but is always higher. This again indicates the conservative nature of the AASHO "C" factor. The AASHO constant is from two percent to 80 percent higher than that obtained from test strains.

TABLE 5.3. SERVICE LOAD DOUBLE AASHO TRUCK DESIGN CRITERIA

Load Location	Girder	K_{GM}	Prototype C	AASHO C
B4-D4, J4-L4	B	1.10	8.2	5.0
	C	1.25	7.2	5.0
	D	0.99	9.1	5.0
	E	0.60	15.0	5.0
	F	0.79	11.4	5.0
	G	1.22	7.4	5.0
	H	1.19	7.5	5.0
	J	1.18	7.6	5.0
	K	1.18	7.6	5.0
	L	1.01	8.9	5.0
B4-D4, E4-G4	A	0.99	9.1	5.5
	B	1.64	5.5	5.0
	C	1.78	5.1	5.0
	D	1.61	5.6	5.0
	E	1.28	7.1	5.0
	F	1.30	6.9	5.0
	G	1.47	6.1	5.0

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The primary purpose of this study was to determine the degree of correlation between the prototype test results and the model test results for pan-formed concrete slab and girder bridges. The secondary purpose was to investigate the feasibility of field testing of this nature. These conclusions are justified by the results of this program:

(1) Reliable methods were developed to determine relative girder live load strain distributions in field tests at service live loads for standard Texas Highway Department pan-formed concrete slab and girder bridges.

(2) Direct reinforced microconcrete modeling techniques, as developed by Leyendecker, are valid procedures for measuring relative girder load distribution. This conclusion is based on the close comparison of model and prototype data.

(3) The AASHO service load distribution factors for moments in longitudinal girders are overconservative when compared to factors developed from field test data. Data for a single

truck load and double truck loads indicate increased AASHO factors are possible.

6.2 Recommendations

(1) Carefully constructed direct microconcrete scale models can be used with confidence to accumulate data beyond the scope of field testing. For instance, a field test to destruction to determine the ultimate strength and ultimate safety factor of this type of bridge was economically unfeasible. Testing of a number of ultimate strength models can be accomplished for the same dollar amount. However, important information concerning service live load behavior can be determined economically in field testing.

(2) The methods developed for strain gage application, load application, and strain data in this field test are adequate for the purpose. Further recommendations for improvements in equipment and test procedure are:

(a) Strain gages should be applied to positive moment steel in all girders, thus eliminating the need for mirror image truck loadings.

(b) Where possible, the reinforcing steel should be shipped from the fabrication shop to the testing base for gage application instead of to the bridge site. Considerable time was used in field application. The cost incurred in transporting the steel to and from the laboratory could be more than made up for in labor saved and improved quality control.

(c) The deflection bridge designed was inadequate for the field test. An underbridge reference wire system, as developed by Breen,[4] seems more advantageous.

(d) From examination of test results at service loads, there is little need for gages other than at the maximum stress location (midpoint of the span). The non-midpoint gages can be used as checks, but serve little other purpose.

(3) A need for more study is indicated in two areas:

(a) Improved techniques for accurate appraisal of dead load strain need to be developed. Since this bridge has a considerable dead load weight, the dead load strains are of an important magnitude. Special interest should be given to the time effects which complicate measurement of dead load strains.

(b) Although each individual span was designed as a simple beam, strains changed as trucks moved on adjacent spans. This indicates continuity of spans. Further tests are indicated for assessment of conditions at the supports.

(4) Although test data indicate the AASHO specifications are conservative, it would be presumptuous to make definite recommendations with the limited data available. Although the guard rails prevent any trucks from moving over the outermost girder, a misguided truck could ultimately end up over this girder. Since the relative stiffness of this girder is less than interior girders, testing should include this girder to make accurate conclusions. Also, the effect that skew angle might have on lateral distribution of load among girders should be investigated.

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A P P E N D I X A

BRIDGE PLANS

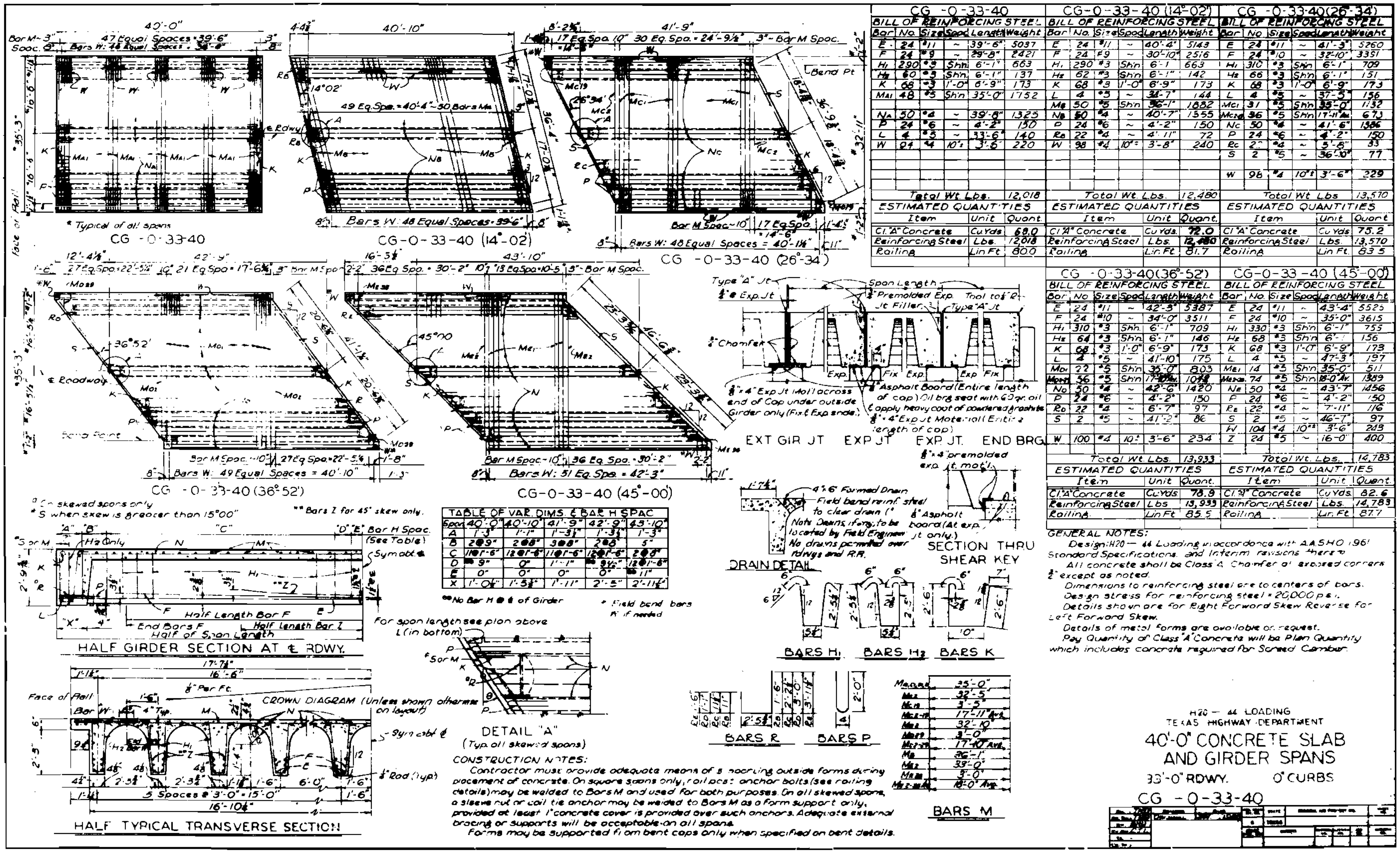

Fig. A.1. Texas Highway Department Plan Sheet, CG-0-33-40.