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JANUARY 1973

Final Report
ISU – ERI – AMES – 72207

FEASIBILITY STUDY OF DYNAMIC OVERLOAD AND ULTIMATE LOAD TESTS OF FULL-SCALE HIGHWAY BRIDGES

Iowa State Highway Commission
Project HR-160

ERI Project 922-S

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W. W. Sanders, Jr. and H. A. Elleby

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Submitted to the
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ABSTRACT

As a result of the construction of the Saylorville Dam and Reservoir on the Des Moines River, six highway bridges are scheduled for removal. Five of these are old high-truss single-lane bridges, each bridge having several simple spans. The other bridge is a fairly modern (1955) double 4-span continuous beam-and-slab composite highway bridge. The availability of these bridges affords an unusual opportunity for study of the behavior of full-scale bridges.

Because of the magnitude of the potential testing program, a feasibility study was initiated and the results are presented in this two-part final report. Part I summarizes the findings and Part II presents the supporting detailed information.

In brief, the following conclusions can be drawn from the study:

(a) for the beam-and-slab bridge:

1. testing to failure is not feasible,
2. dynamic testing at design load and overload levels will provide useful data, and
3. testing of deck components under static and fatigue loads should be conducted.

(b) for the high-truss bridges:

1. ultimate load tests should be conducted on three selected spans,
2. fatigue tests should be undertaken on complete component members selected from all truss bridges, and
3. tests should be conducted on in-place timber decks and timber stringers.

Study results show that significant information on the behavior of bridges designed for normal service can be obtained from a wide variety of tests. An outline of these tests is presented.

PART I—SUMMARY OF FINDINGS

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INTRODUCTION

As a result of the construction of the Saylorville Dam and Reservoir on the Des Moines River, six highway bridges are scheduled for removal. Five of these are old high-truss single-lane bridges, each bridge having several simple spans. The other bridge is a fairly modern (1955) double 4-span continuous beam-and-slab composite highway bridge. The availability of these bridges for overload and destructive tests presents an unusual opportunity for studying the behavior of bridges.

Because of the magnitude of the potential testing program a contract was awarded to the Engineering Research Institute of Iowa State University by the Iowa State Highway Commission to conduct a study of the feasibility of a field investigation of dynamic properties and ultimate load capacity of the bridge superstructures. The results of that feasibility study are presented in this two-part report. Part I summarizes the findings and Part II presents the supporting detailed information.

PROGRAM OF STUDY

The purpose of this investigation was to study the feasibility of conducting dynamic overload and ultimate strength tests on the six highway bridges to be removed from the Saylorville Reservoir, to propose a general testing program in line with the conclusions, and to investigate possible sources of funding and cooperative efforts.

In order to fulfill these objectives, a general working plan was developed. In brief, it included:

- a review of all available literature on related bridge tests,
- a field investigation of the considered bridges to determine the physical condition of the structure and the site,
- an analysis of the bridges under various loading conditions to determine anticipated behavior under field loadings,
- preparation of a proposed testing program considering the bridge analysis research needs, site conditions, construction timetables, and economic factors,
- contacts with agencies interested in cooperating in the testing phase of this investigation,
- preparation of a final report on Phase 1 (feasibility study) of the investigation.

This part of the final report will summarize the results of this investigation, whereas Part II will deal primarily with the details of the bridge analyses. The following sections will deal with each of the above portions of the working plan.

BACKGROUND ON FIELD TESTING OF BRIDGES

In the last 25 years a considerable number of field tests[1] on bridges have been conducted. Nearly all of these were conducted at or near design loads.

However, with approval by the American Association of State Highway Officials (AASHO) of load factor design[2,3] for steel bridges, considerable interest has been generated in tests of actual steel bridges at overload levels and to failure. A very limited number of these tests have been conducted; most were performed either on laboratory models or on specially designed bridges, such as the AASHO Road Tests[4,5]. The exceptions are a 1960 test of the Glatt Bridge in Switzerland[6] and four tests recently completed in Tennessee[7-10]. In addition, a special test is planned for summer of 1973 on a bridge in Southeast Missouri[11].

The tests conducted as a part of the AASHO Road Tests[4,5] were made on eighteen 50-ft simple-span single-lane beam-and-slab bridges. These bridges, which consisted of slabs supported by reinforced concrete, prestressed concrete or steel beams, were specifically designed for the test program. Although providing valuable information, the results did not truly indicate the behavior of normally designed bridges. The bridge tested in Switzerland[6] was a prestressed concrete rigid frame bridge, and is not typical of current design practice in this country.

The most significant contribution to current knowledge of overload and ultimate strength behavior of bridges resulted from the four tests conducted by the University of Tennessee[7-10] in conjunction with a research study for the Tennessee Department of Highways and Federal Highway Administration. The four highway bridges were first tested dynamically using a standard AASHO design truck, an overloaded highway truck, and an Army tank transporter. The bridges were then tested to failure using simulated truck loads. One of the bridges, a 4-span continuous steel beam-and-slab bridge, was similar to the beam-and-slab bridge in the Saylorville area.

As noted, the University of Missouri at Columbia[11] is under contract to study the behavior of a 3-span continuous composite highway bridge. This study, conducted for the Missouri State Highway Department and the Federal Highway Administration, includes fatigue tests of the bridge to evaluate shear connectors and girder cover plates and an ultimate load shakedown test of the center half of the bridge. These tests, although related to the studies considered herein, are not directly applicable.

Several additional research organizations are investigating the possibilities of conducting overload and ultimate strength tests of actual bridges, but as yet, no such studies have been formalized.

Thus, the information available on overload and ultimate behavior of actual bridges is very limited and, even then, is limited to beam-and-slab type bridges.

DESCRIPTION OF BRIDGES STUDIED

The six highway bridges are located on the Des Moines River immediately northwest of Des Moines, Iowa, in an area which will be included in the Saylorville Reservoir. The Reservoir is being constructed by the U.S. Army Corps of Engineers (North Central Division — Rock Island District). The project is scheduled for completion in June 1975.

The prime function of the reservoir will be to store floodwaters of the Des Moines River. As a result there can be a considerable fluctuation in the lake level. The differential between the conservation and flood pools is about 57 ft. As a result, several of the bridges being studied will be inundated only during a flood while the remainder will be covered (at least partially) by the conservation pool.

The six bridges consist of five high-truss single-lane simple-span bridges and one 8-span (double 4-span) continuous beam-and-slab bridge. The five high-truss bridges were built about 1900, thus information on their design and construction is limited. The beam-and-slab bridge, however, was built about 1955 and extensive design and construction data are available from the Iowa State Highway Commission.

Three of the high-truss bridges are located in Polk County, while the others are in Dallas and Boone Counties. A brief description of each bridge follows.

Corydon Bridge: The Corydon Bridge (Figs. I-1 and I-2) is located about 3 miles above the dam in Polk County, just south of Polk City, and carries County Road S. The bridge, built in 1889, includes two Pratt type high-truss simple-span truss bridges. It has a 16-ft roadway, with each span being 156 ft long with a height of 24 ft 6 in. The floor system consists of steel I-beams for the floorbeams with timber stringers and deck.

Hanley Bridge: The Hanley Bridge (Figs. I-3 — I-6) is located about 7 miles northwest of the dam in Polk County and is immediately east of Jester County Park. The bridge, which carries County Road V, originally consisted of three pin-connected high-truss spans; however, about 20 years ago, two of the spans were damaged and removed. Two riveted trusses (standard ISHC T-series) were moved from the Skunk River and erected at the east end. The three spans, each with a 16-ft roadway, include a 150-ft pin-connected Pratt type high-truss with steel floorbeams and timber stringers and deck, a 110-ft riveted truss with a concrete deck (ISHC-T5), and a 100-ft riveted truss with a concrete deck (ISHC-T3).

Snyder Bridge: The Snyder Bridge (Figs. I-7 and I-8) is located in northwest Polk County near the northwest corner of Jester County Park. Prior to being closed recently, it carried County Road X. The bridge, built in 1898, consists of three modified Pratt pin-connected high-truss spans with a 16-ft timber deck roadway. The deck is supported by steel floorbeams and timber stringers.

Chestnut Ford Bridge: The Chestnut Ford Bridge (Figs. I-9 — I-11), built about 1900, is located in northeast Dallas County (Des Moines Township) and is about 13 miles above the dam. It is composed of four simple-span pin-connected high-truss bridges. As shown in Fig. I-11, the two western spans and the eastern span are 150 ft long and the other span is 180 ft long. At the east end of the 180-ft span, there is a 9° angle in the roadway. The roadway is typical of other bridges in the reservoir area.

Hubby Bridge: The Hubby Bridge (Figs. I-12 and I-13) is located in southern Boone County about 5 miles southwest of Luther. Built in 1909 the bridge is composed of four modified Parker type high-truss simple spans, each 165 ft long. The deck is built of timber stringers and decking supported by steel floorbeams. The stringers in the east two spans are creosote treated, whereas the west spans are not.

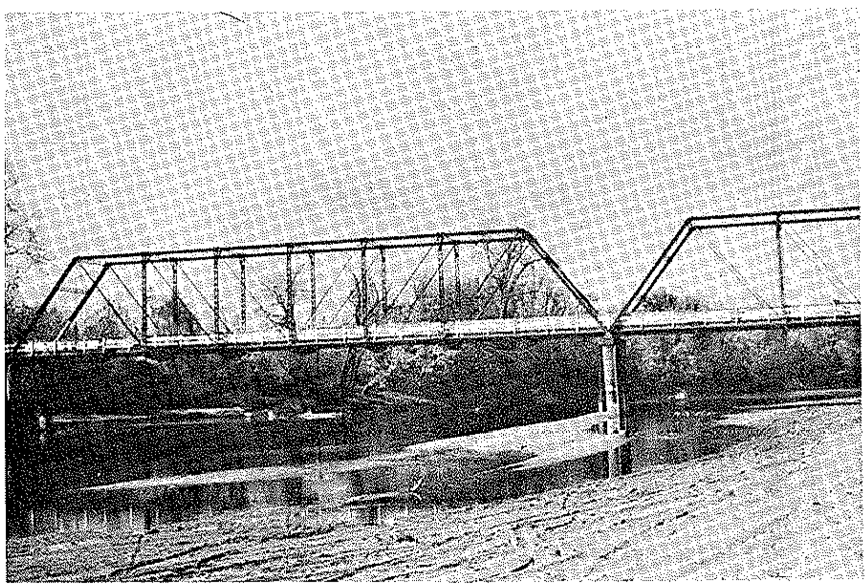

Fig. I-1. Photograph of Corydon Bridge (Polk County).

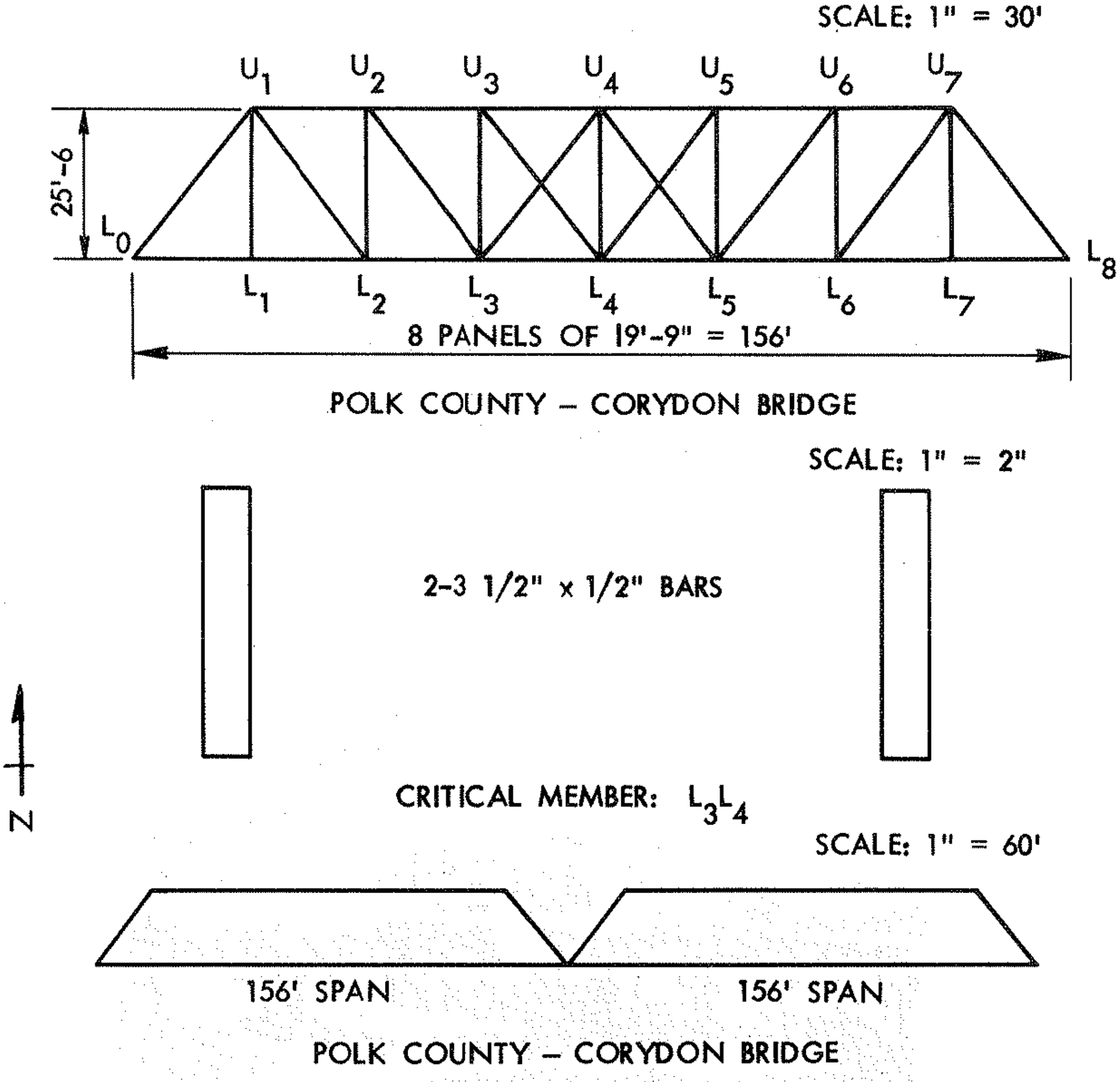

Fig. I-2. Details of Corydon Bridge (Polk County).

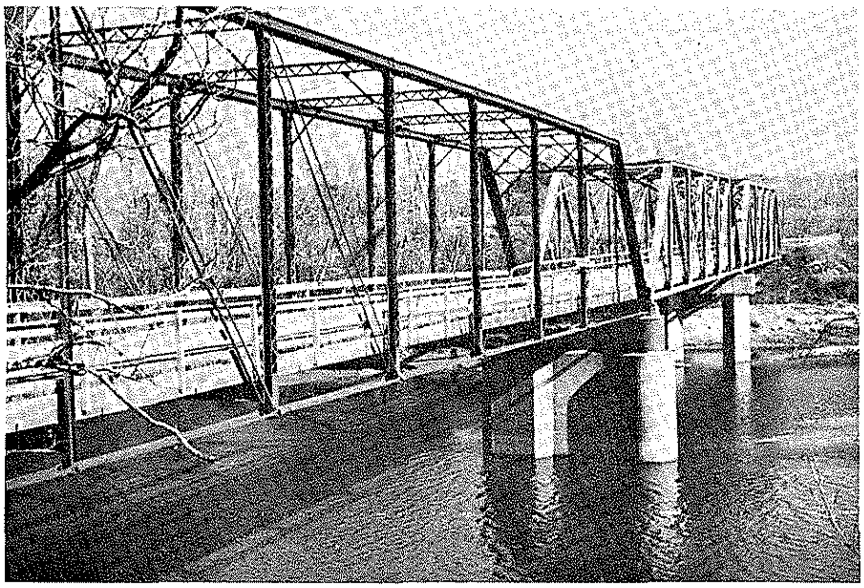

Fig. I-3. Photograph of Hanley Bridge (Polk County).

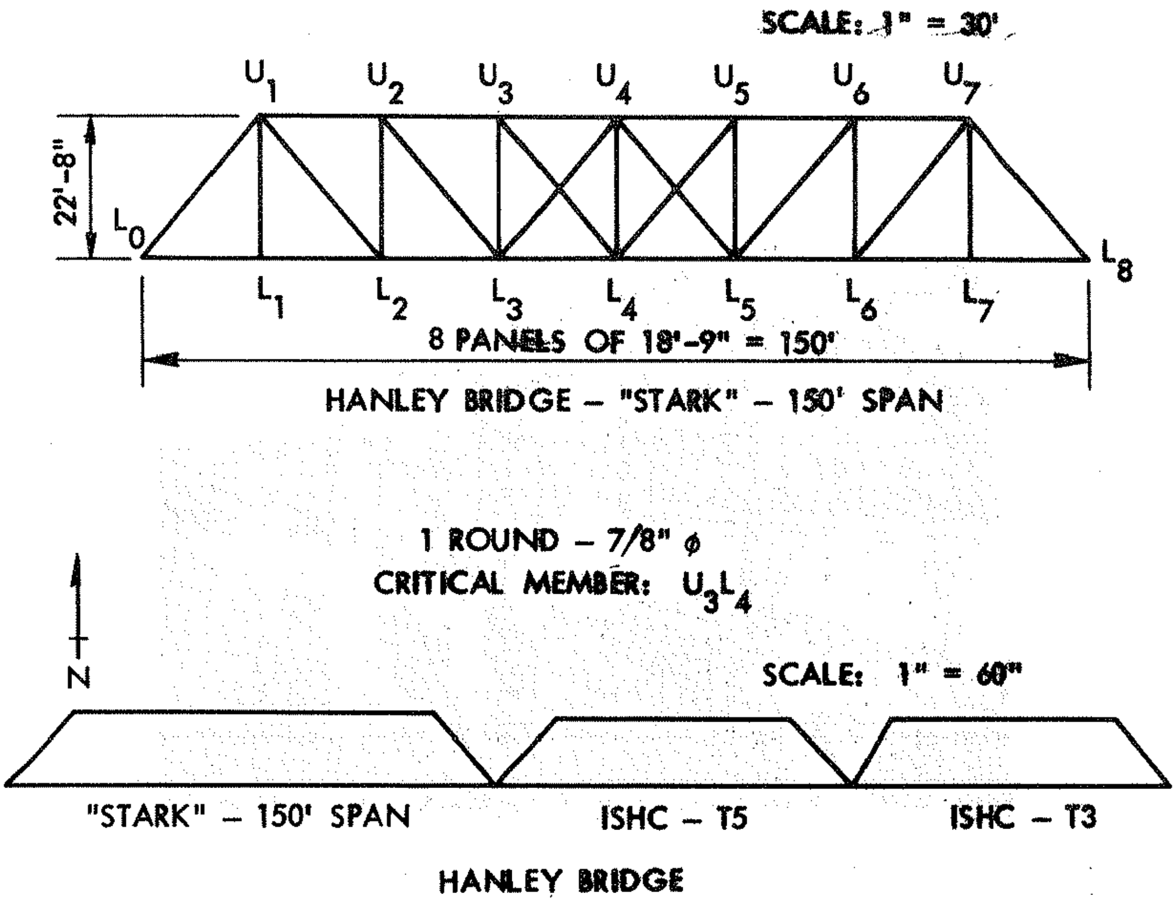

Fig. I-4. Details of Hanley Bridge (Polk County) — "Stark" 150-ft span.

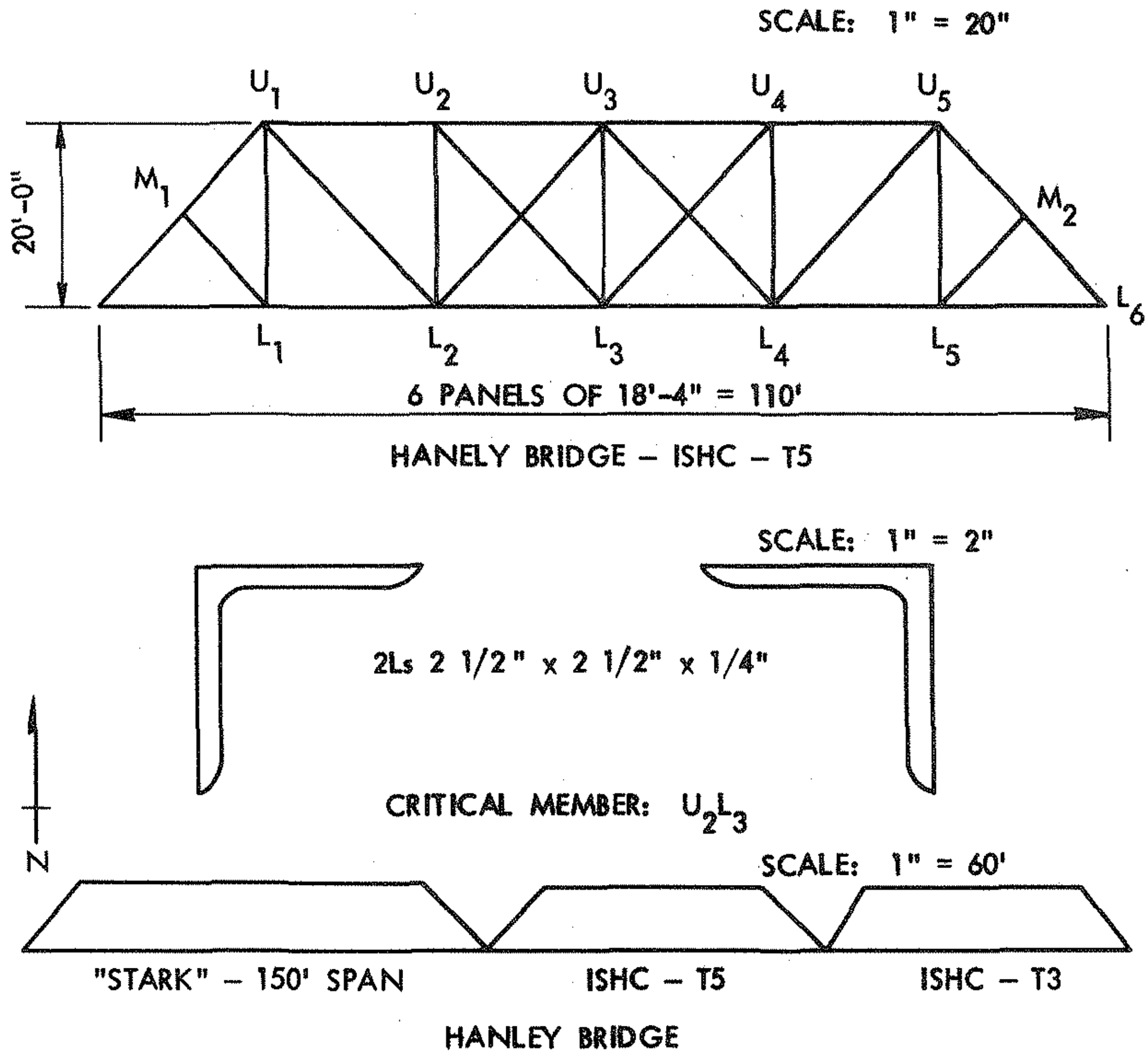

Fig. I-5. Details of Hanley Bridge (Polk County) — ISHC-T5 span.

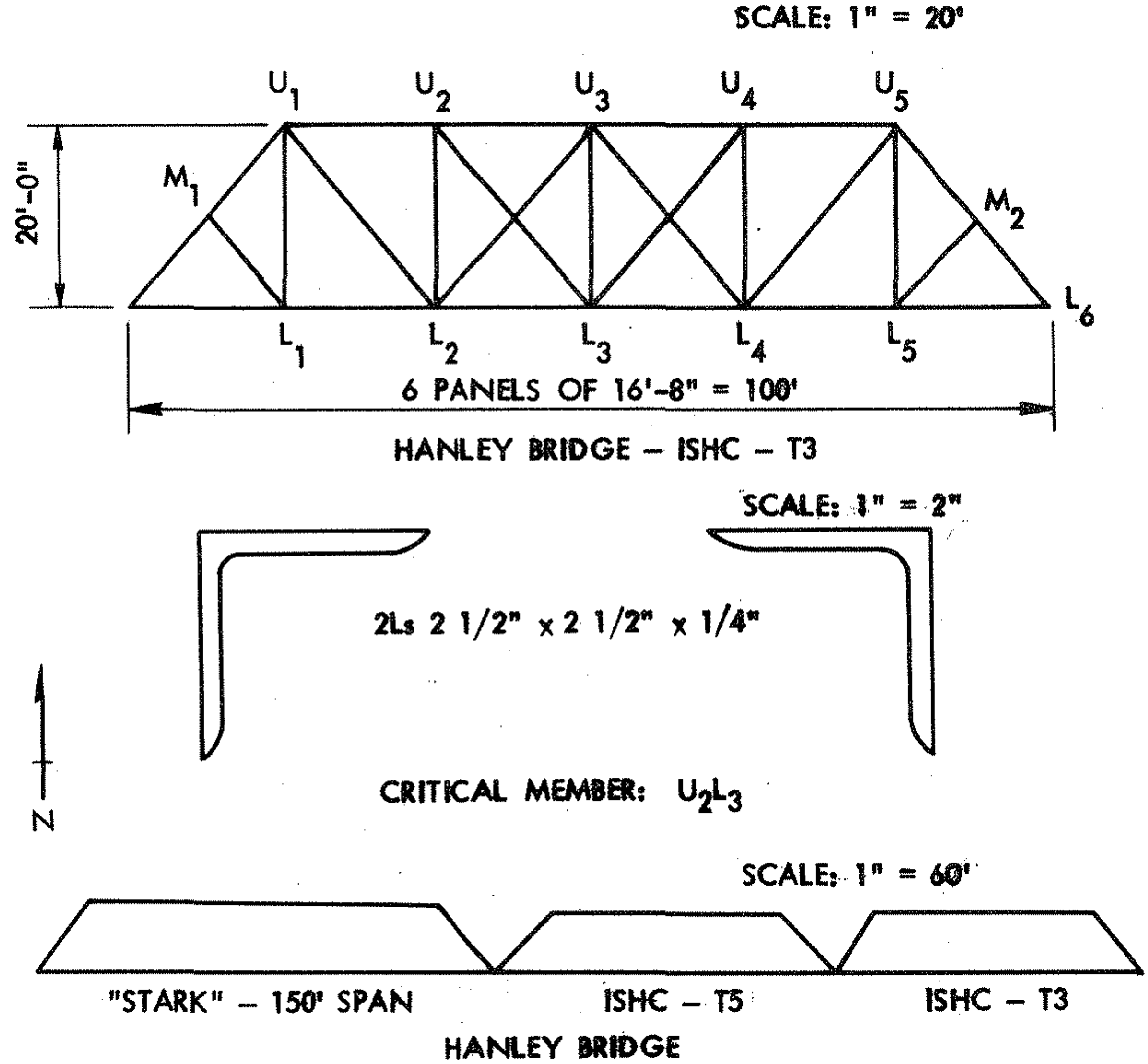

Fig. I-6. Details of Hanley Bridge (Polk County) — ISHC-T3 span.

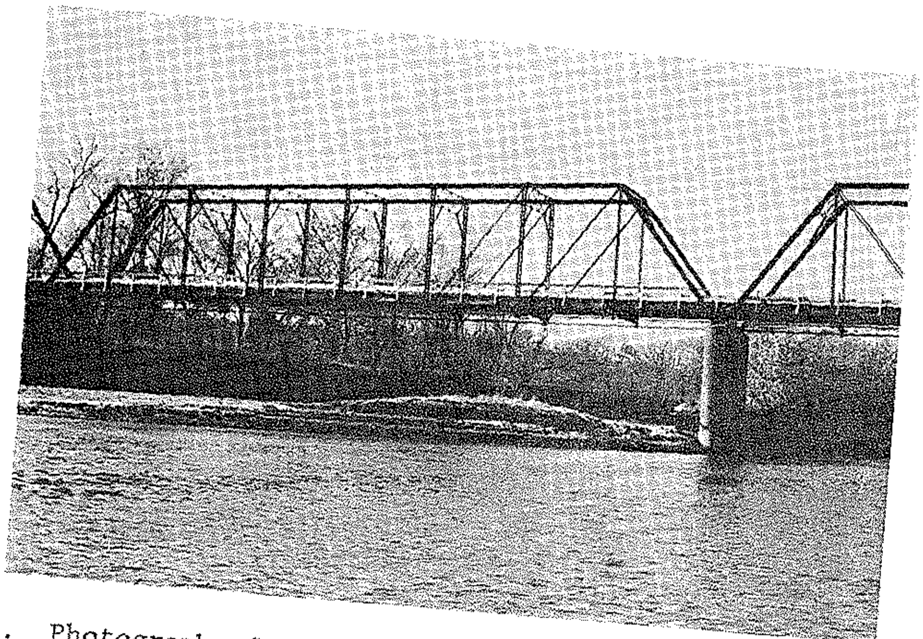

Fig. I-7. Photograph of Snyder Bridge (Polk County).

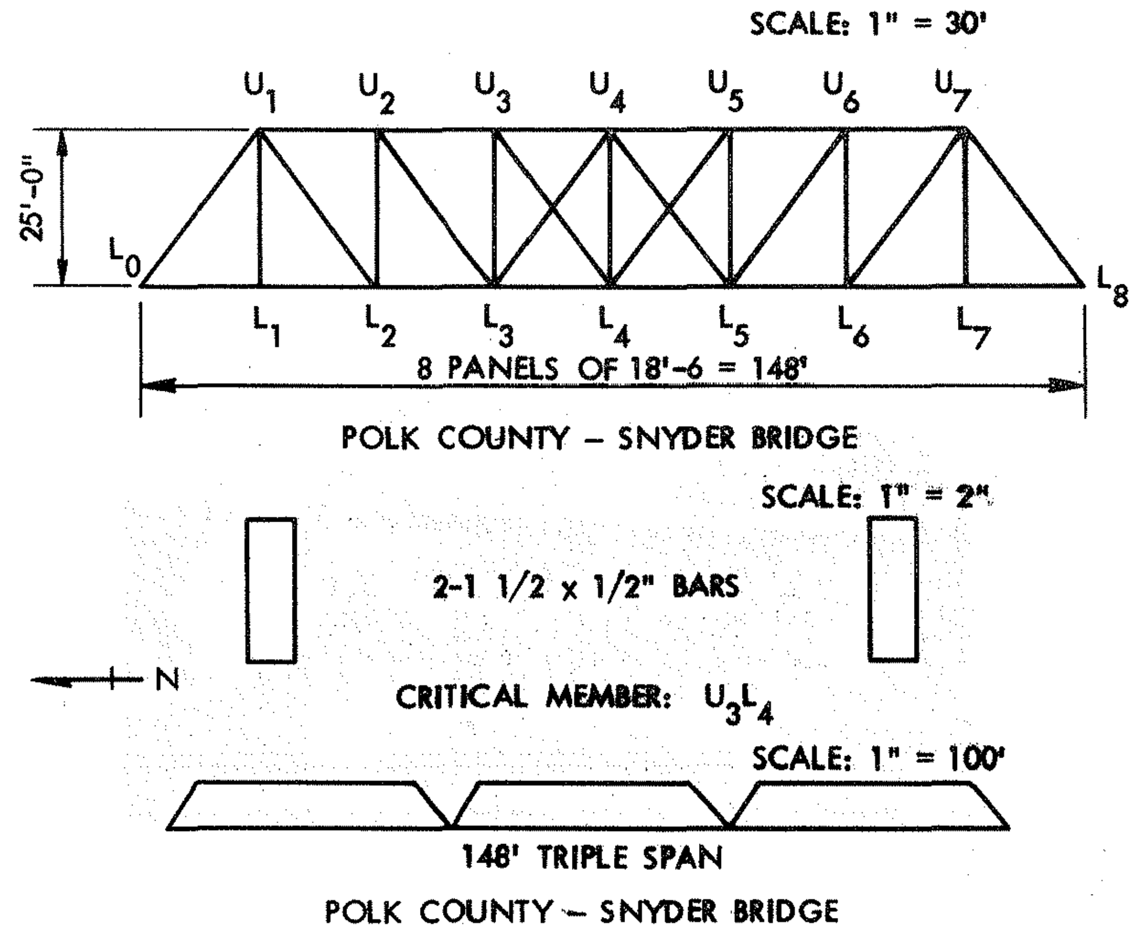

Fig. I-8. Details of Snyder Bridge (Polk County).

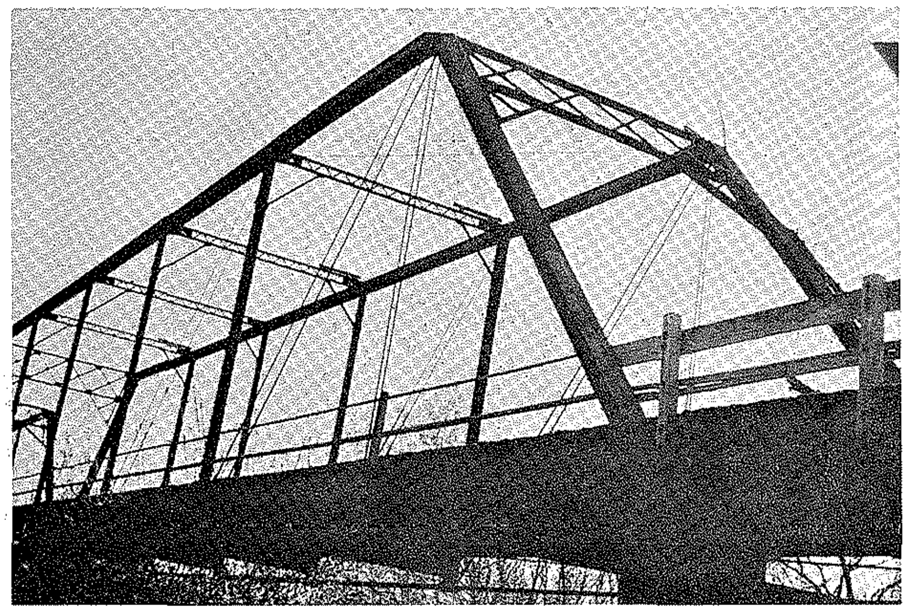

Fig. I-9. Photograph of Chestnut Ford Bridge (Dallas County) — 150-ft span.

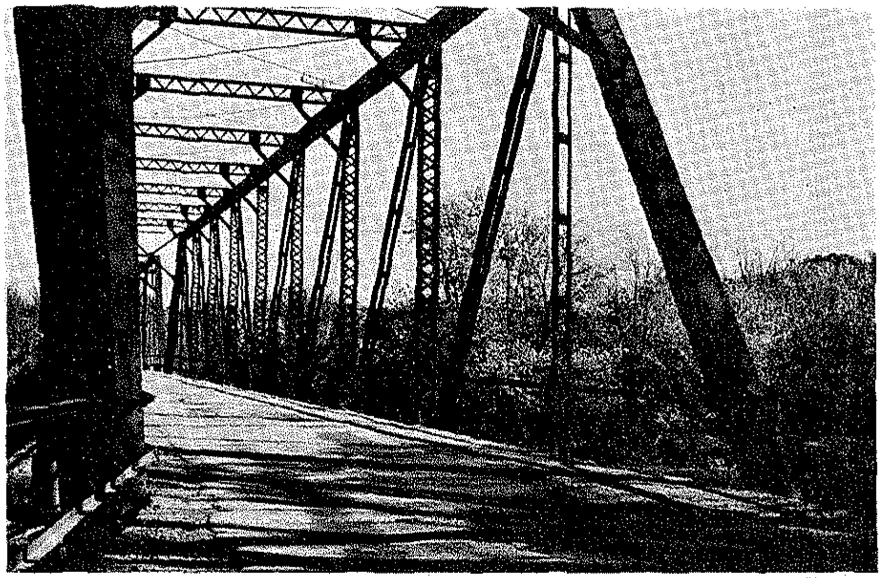

Fig. I-10. Photograph of roadway of Chestnut Ford Bridge (Dallas County) — 180-ft span (viewed from adjacent 150-ft span).

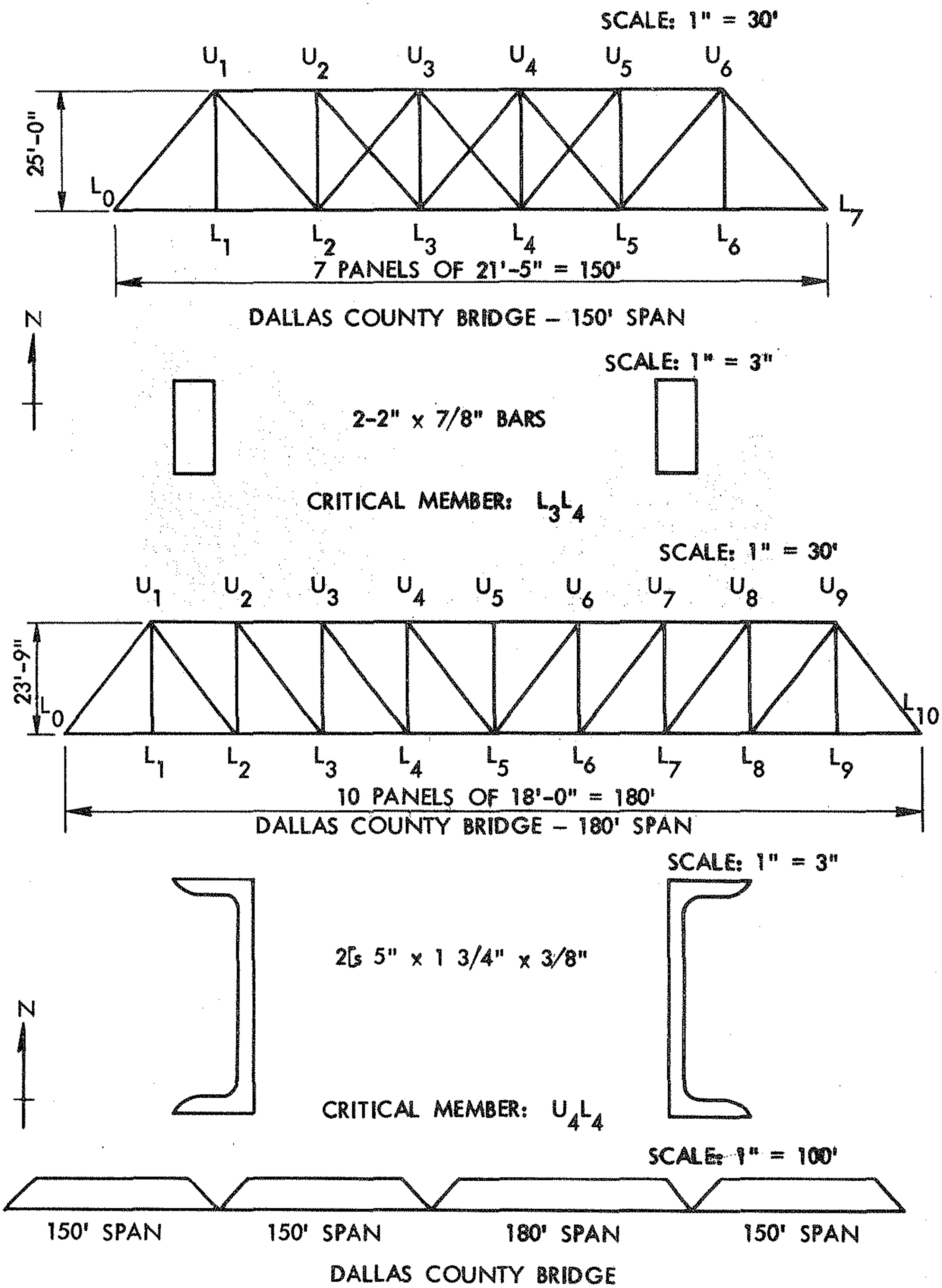

Fig. I-11. Details of Chestnut Ford Bridge (Dallas County).

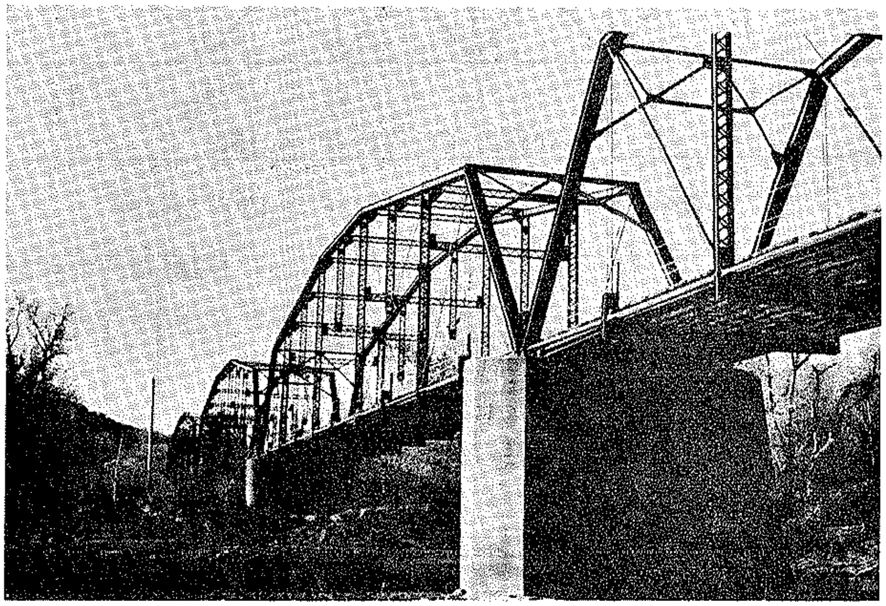

Fig. I-12. Photograph of Hubby Bridge (Boone County).

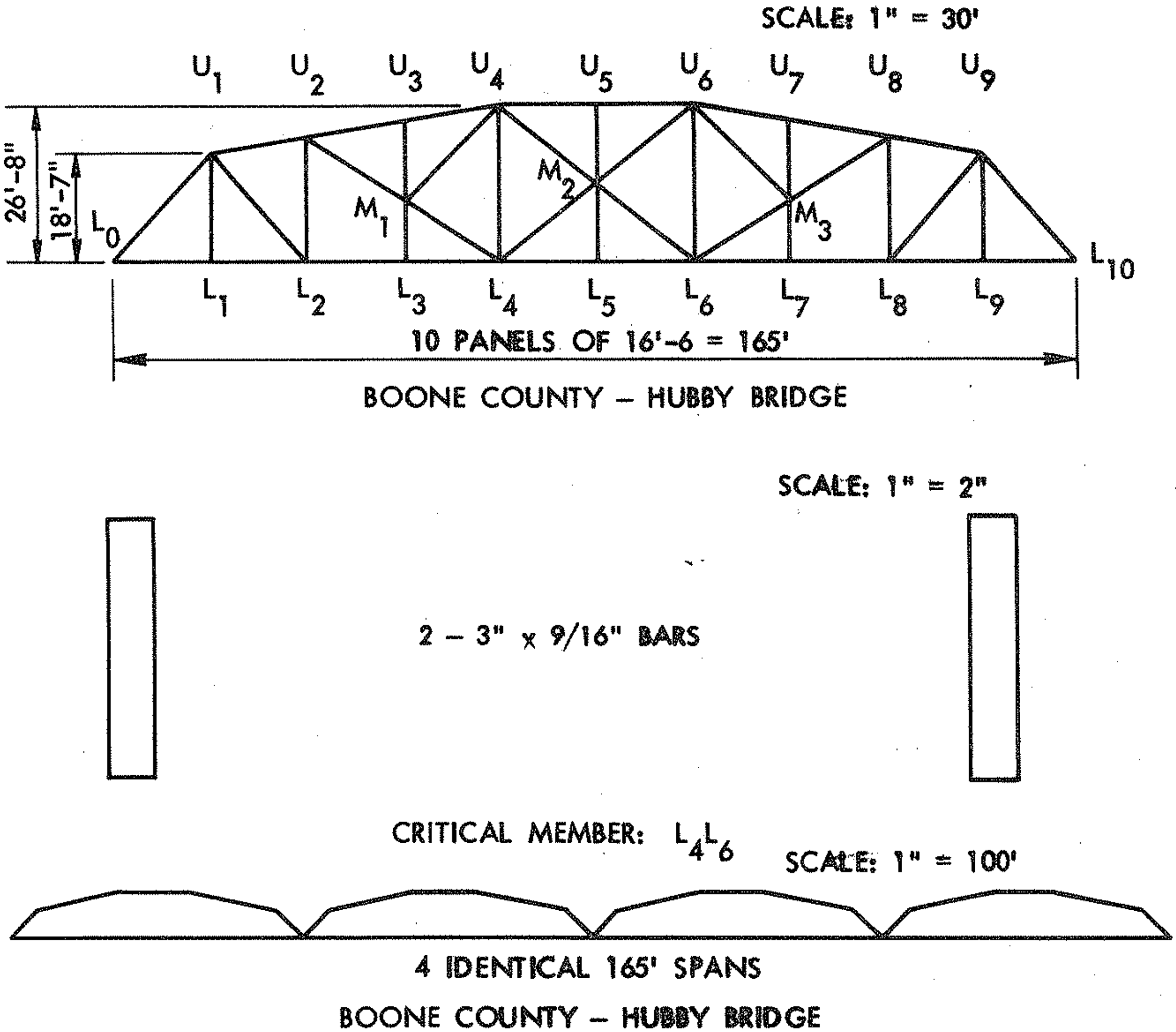

Fig. I-13. Details of Hubby Bridge (Boone County).

The major bridge being studied in this investigation is the 723 X 26 ft beam-and-slab bridge crossing the Des Moines River between Madrid and Woodward. This bridge (Figs. I-14 to I-16) is located in extreme southern Boone County. It was built about 1955 and consists of a 306-ft 4-span continuous approach bridge and a 417-ft 4-span continuous main bridge. The approach bridge is on the west end and is situated over river bottom land, which is under water only during flood stage. The main span (on the east) is primarily over the Des Moines River channel.

Because of the need, or desirability, to have access to the under side of the superstructure and to have a stable surface from which to locate and install instrumentation, the primary thrust of this study was directed to the approach spans. In these spans the general ground line (Fig. I-16) is 12 to 16 ft below the main steel beams. The approach bridge is built of four rolled steel beams (interior-36WF150 and exterior-33WF130) spaced at 8 ft 3 in. with welded cover plates over the piers and with bolted diaphragms at 22-ft intervals. The concrete deck, including curbs is 32 ft wide with a 26-ft roadway of nominal 7-in. reinforced concrete.

One of the critical considerations in the study of the feasibility of ultimate load tests is the soil conditions at the site. An examination was made of ISHC borings taken at the site just prior to construction of the bridge and of Corps of Engineer borings taken recently at the site of the adjacent new bridge and the new railroad bridge about 1000 ft to the south. These borings indicate that the subsurface is generally composed of about 15 ft of fine sand and lean clay, about 20 ft of gravelly sand underlane by shale and fissile with small limestone nodules. Details of the soil conditions at the site are presented in Part II of this report.

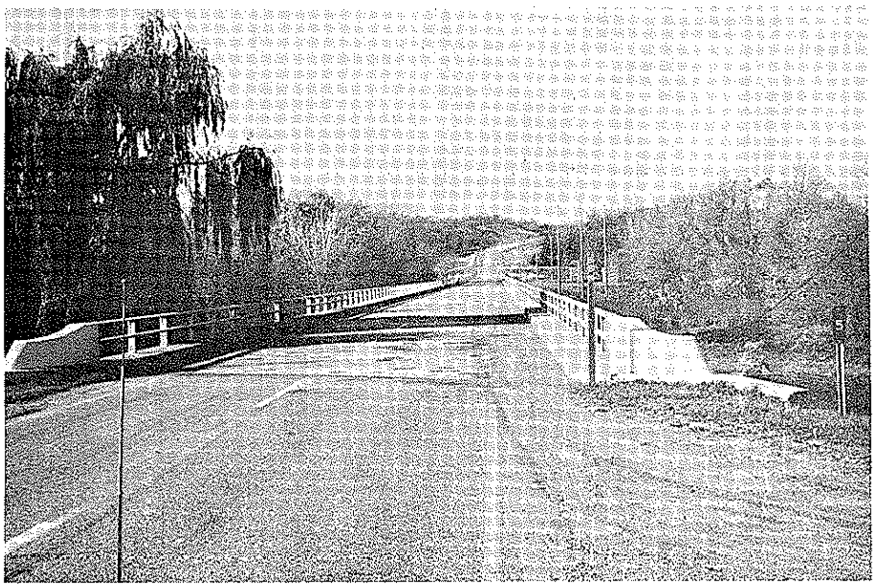

Fig. I-14. Photograph of Iowa 89 Bridge (looking west toward Woodward).

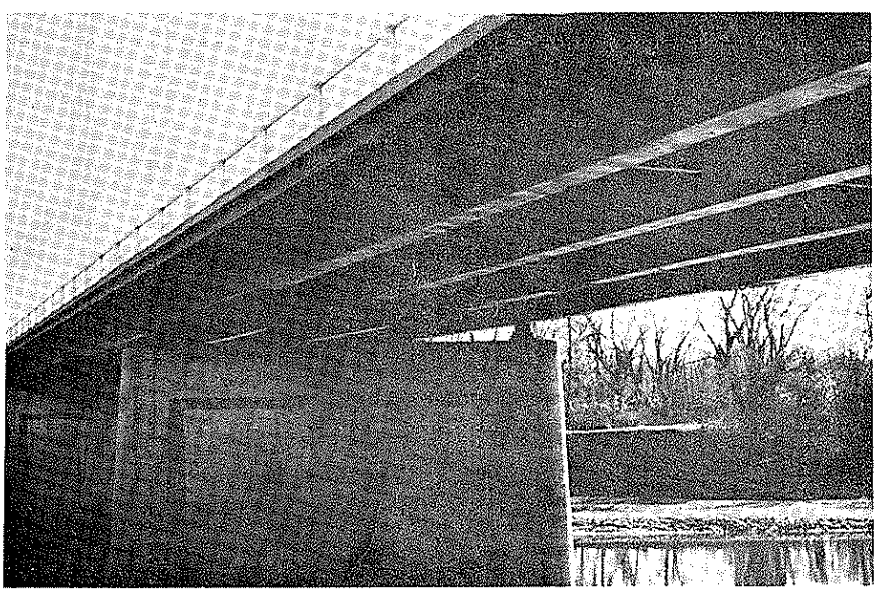

Fig. I-15. Photograph of approach span and portion of main span (looking south).

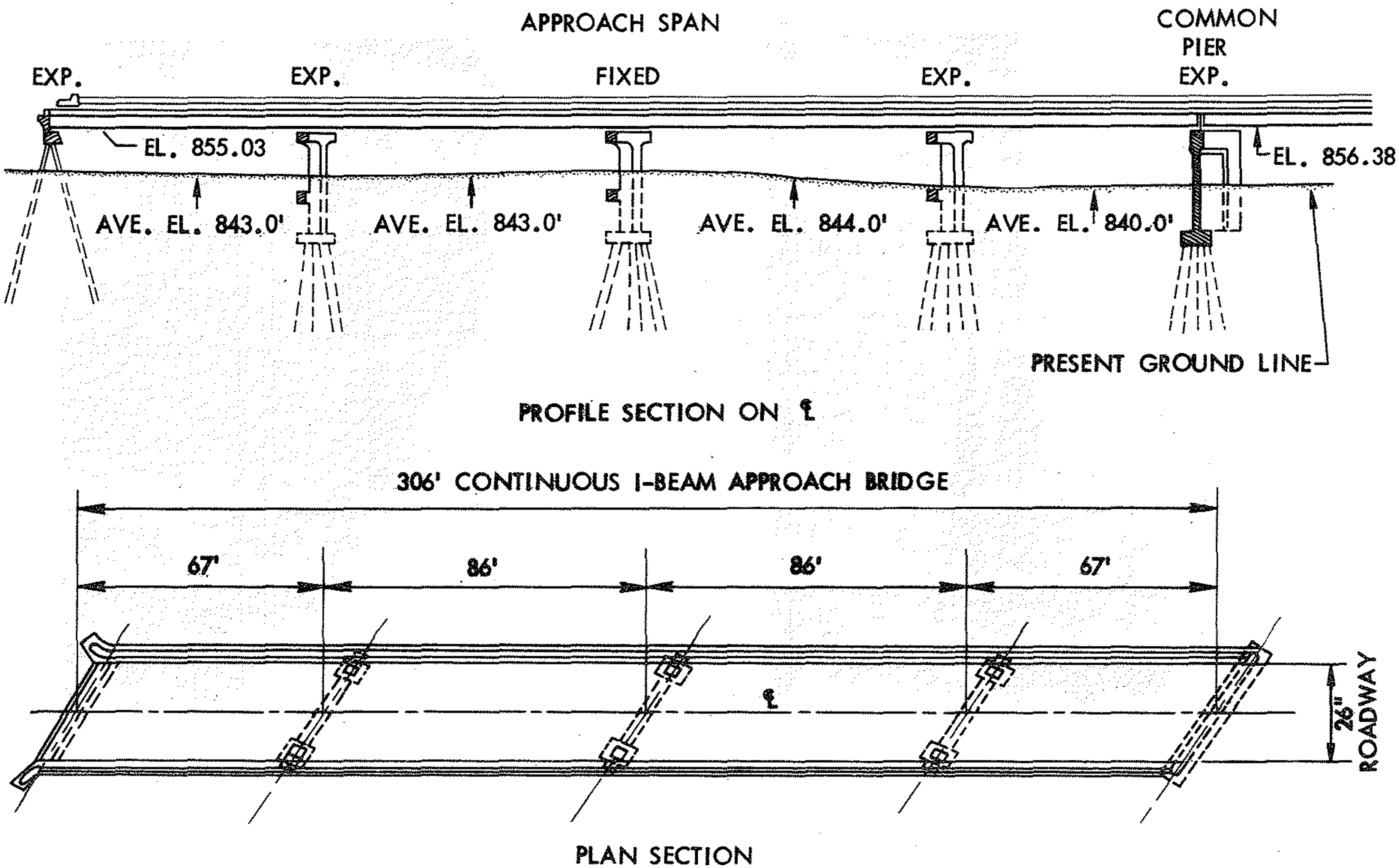

Fig. I-16. Profile and plan of approach bridge (Iowa 89).

RESULTS OF BRIDGE ANALYSIS

As noted earlier there was considerable information available on the Iowa 89 Bridge, whereas only sketchy data were available on the high-truss bridges. The first step in analyzing the high-truss bridges was to determine the configuration and strength of the truss members. Notes were made on member deterioration and on the sections and properties of the floor system. With this information available, all six bridges in this study were analyzed.

High-Truss Bridges

The original designs of the high-truss bridges were probably based on a single 15-ton engine. This engine consisted of a 20,000-lb rear axle and a 10,000-lb front axle. In order to simulate this loading, each truss was analyzed considering it to be loaded with either a single 15-kip load at midspan, if a panel point occurred at midspan, otherwise a pair of 10-kip loads at adjacent center panel points. These loads were used because it was felt that the floor system would not sustain a load as large as the trusses and, therefore, the failure loads would be applied directly to the trusses.

The trusses were analyzed, using a standard computer program, to determine bar forces. It was assumed that in panels where counters were present, only the tension member was effective. It was also assumed that failure occurred in tension members when the stress reached the ultimate strength (assumed 60 ksi), although the yield strength (33 ksi) was noted, and in compression members when the stress reached the buckling stress (as suggested by the American Institute of Steel Construction). The ultimate loads are given in Table I-1.

These loads were found by determining the number of base loads (15 or 20 kips) that were required to fail each member. The member with the lowest load increment (LI) would fail first. The members were

Table I-1. Ultimate loads on truss bridges.

Bridge/span	Critical member	Base load/LI[a] (kips)	Ultimate load (kips)/truss
Corydon	L_3L_4 (y)[b]	15/4.3	65
	U_2L_2 (u)	15/5.9	89
Hanley			
Stark span	U_3L_4 (y & u)	15/1.97	30
110-ft (T5)	U_2L_3 (y & u)	15/13.2	198
100-ft (T3)	U_2L_3 (y)	15/5.1	102
	U_2L_3 (u)	15/7.7	116
Snyder	U_3L_4 (y & u)	15/9.3	140
Chestnut Ford			
150-ft span	L_3L_4 (y)	20/3.6	72
	U_2L_2 (u)	20/5.2	104
180-ft span	U_4L_4 (u)	15/7.5	113
Hubby	L_4L_6 (y)	15/2.1	32
	L_4M_5 (u)	15/3.8	57

[a] LI = load increment; the number of multiples of the base load to cause yielding or ultimate.

[b] y = critical at yield; u = critical at ultimate.

assumed to be pinned at each end, although it is fully realized that some rigidity exists either from construction or deterioration and rusting.

Iowa 89 Bridge

In determining the longitudinal behavior of the bridge, it was felt that predictions accurate within 10 to 20% would be sufficiently accurate for purposes of the feasibility study. Although a summary is included in this portion of the report, a detailed presentation of these results will be found in Part II.

The moments in the bridge at loadings below the yield level of the bridge were found using influence coefficients provided by the Iowa State Highway Commission's Bridge Design Department. The load distribution of the truck wheel loads on the bridge was found by two methods. The first method determined the transverse position of the trucks using an influence line assuming nonelastic beam supports to the slab. The second method employed orthotropic plate theory and harmonic analysis. A rational value obtained from the two results was used to determine the girder moments and stresses.

For ultimate load, which would be conducted using anchor rods or a similar system, the front wheels of the trucks were considered absent and only the main loading from the truck and trailer rear wheels was used (eight concentrated loads). To maximize the moment, the truck was positioned on the bridge with the bridge centerline 3.5 ft from one axle and 10.5 ft from the other. A plastic analysis was conducted on the interior span assuming a mechanism to have formed. One plastic hinge was assumed at the wheel load nearest the midspan with the other two hinges forming in the girders at the end of the cover plate beyond the piers in adjacent spans. A summary of the results is given in Table I-2. The end spans were not analyzed because it was felt that the test loading would be more productive in the interior spans.

Table I-2. Results of ultimate load analysis of Iowa 89 Bridge.

Description	Equivalent truck loading	Single conc. load[a]
Interior girder yield of bottom flange	HS-44 (2-centered)	35 kips
Exterior girder yield of bottom flange	HS-70 (1-eccentric)	56 kips
Interior and exterior yield of bottom flange	HS-77 (2-centered)	61 kips
Ultimate loading	HS-130 (2-centered)	105 kips

[a] Single wheel load equivalent to HS loading given.

From Table I-2 it can be seen that two vehicles of more than double the standard loading could drive down the bridge at crawl speed without yielding the bridge. This does not include the dynamic effects due to impact or imperfections on the bridge. It is at this stage, however, that strength of the slab and load distribution contribution of the diaphragms has the greatest influence on the bridge behavior.

The deflection of the bridge at ultimate was estimated to be 2.43 ft. This was determined using the moment curvature relationships that exist in the bridge at ultimate loading.

The fundamental frequency of the bridge was estimated to be about 3 cycles/sec. This was determined using estimates of the equivalent stiffness of the bridge and an equivalent mass system with a single truck and applying these results to a single degree of freedom system. These results are comparable to measured frequencies of existing bridges.

In order to properly estimate the behavior of the bridge system as a whole, the lateral or transverse load carrying capacity must also be determined. To estimate the transverse loading capacity of the slab, three methods were used. It was first assumed that the effective width of the slab was 16 times its thickness. This would be 9.33 ft longitudinally along the bridge. Using this estimate of 9.33 ft, the ultimate load would be 76 kips and approach the upper bound. A more realistic estimate of this effective length uses the theory of elasticity and would be around 0.6 of the distance between girders or about 5.0 ft. The estimate of 5 ft would lead to lower bound value for the loading capacity of the slab. With the load located midway between girders, this would be 28 kips. The value found using yield line theory was 73 kips[*].

The maximum moment in the slab at midspan would be 1.65P to 1.98P ft-kips in the elastic region, where P is the applied load. It should be noted, however, that the maximum shear capacity for a 12-in. diameter circle is 55 kips. Therefore, if substantial overloads or ultimate loads are used, punching shear must be considered.

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[*]Thus the actual capacity of the slab will fall someplace between the bounds of 28 and 76 kips.

The fundamental frequency of the slab was estimated at 15 cycles/sec using the same assumptions as used to determine the fundamental frequency of the bridge. The stresses in the slab produced by an excitation force would then be a 15 cycle/sec periodicity superimposed on the 3-cycle frequency response of the bridge acting as a whole.

CONTACTS WITH OTHER ORGANIZATIONS

One phase of this study included contacting other organizations and agencies to determine:

- current research in related fields,
- design and construction information on structures in the Saylorville Dam area,
- potential sponsorship or cooperation in testing program.

Agencies or organizations contacted in relation to the above objectives included:

- Rock Island District, North Central Division, U.S. Army Corps of Engineers;
- Iowa State Highway Commission (ISHC);
- County Engineers, Boone, Polk and Dallas Counties;
- Office of Research and Development (Bridge Division), Federal Highway Administration (FHWA);
- Department of Civil Engineering, University of Tennessee, Knoxville;
- Department of Civil Engineering, University of Missouri, Columbia;
- International Road Federation (IRF);
- American Trucking Association (ATA) and Iowa Motor Truck Association (IMTA);
- U.S. Army — Military Traffic Management and Terminal Service (MTMTS) and Tank Automotive Command (TACOM).

The information from the Corps of Engineers, the Iowa State Highway Commission and the County Engineers related primarily to design details of current and proposed bridges and construction timetables.

Discussions with the Federal Highway Administration (FHWA) and the two universities concerned potential research programs and their relationship to present research, although FHWA cooperation in the field study was discussed. Contacts with the other agencies (IRF, ATA, IMTA, MTMTS and TACOM) dealt with possible assistance in funding. Although not contacted about any of the aforementioned phases, numerous area contractors and suppliers were contacted concerning construction procedures and costs, as well as equipment costs.

Much of the data received dealt directly with properties of the bridges studied and the reservoir site and related research conducted or underway. This information was used in the analyses and background information previously discussed. The remaining information can be briefly summarized in three categories: construction schedule, cost estimates, and potential funding and cooperation. Comments on the effect of the information is included.

Construction Timetable

One of the critical considerations in determining the research which can be conducted is the availability of the bridges for testing. The construction timetable for the new bridges and the dam control this availability.

Based on information currently available, this timetable is:

November 14, 1972: Contract letting for new Iowa 89 bridge (removal under separate contract).

December 1972: New Hubby Bridge accepted as substantially complete and opened to traffic (old Hubby Bridge could be closed as soon as final painting on new bridge is completed in Spring 1973).

November 30, 1974: Contract termination date for new S and V bridge (replacement for Corydon and Hanley Bridges); however, contractor is attempting accelerated construction program with completion estimated as early as November 30, 1973.

November 30, 1974: Contract termination date for new Iowa 89 Bridge.

Summer and Fall 1974: Bridge removal contracts. It is expected that two and possibly three contracts will be awarded. One will be for the present Iowa 89 Bridge with the others for the high-truss bridges. These contracts cannot, however, be initiated until completion of the new bridges. With the completion dates being very near the date when the dam is operational (flooding possible), removal may have to begin immediately.

January 1, 1975: Dam is to be operational.

June 30, 1975: Scheduled completion date for entire project.

It can readily be seen that the construction timetable will significantly affect the length of time that at least three bridges (Iowa 89, Corydon and Hanley Bridges) are available for dynamic or failure testing. Since all testing probably will have to be completed before November 1974, the testing of these three bridges may be severely limited unless early completion occurs.

Since the Snyder Bridge is already closed and the old Hubby Bridge will be replaced about May 1973, these bridges may be available for testing in late Summer 1973. Furthermore, the Chestnut Ford Bridge (Dallas County) is not being replaced and could also be available earlier. Its availability, however, is questionable since it is the only access from most of the county, including the county seat, to a small area (about 5 square miles) in the northeast corner of the county.

Cost Estimates for Bridge Salvage or Removal

One consideration in evaluating proposed tests is the cost of removing bridges if tested to failure and the loss in salvage value due to any overloading (inelastic behavior).

The removal of the high-truss bridges would probably be accomplished by removing the timber floor system, dropping the truss in the river channel or bottom land, dragging the truss to dry land (if necessary), and removing to a scrap or salvage yard. It can be seen, therefore, that the loss of deck salvage resulting from destructive testing of the timber floor system will be the only significant cost increase involved in a study of the high-truss bridges. In addition, a slight savings may result from partial destruction of the trusses from any ultimate load tests. It is estimated that removal of the superstructure for each bridge will average about $2,500 per span with an additional $1,000 for removal of each pier (about $2,000 if the pier is steel jacketed).

Removal of the Iowa 89 Bridge will probably be completed by removing the concrete deck, dismantling and salvaging the bolted steel superstructure, then destroying the piers and abutments. If sufficient loading was applied to the steel superstructure to cause permanent deformation, it is doubtful if any salvage could be secured from the deformed spans.

The direct monetary loss, however, would be insignificant since only one or two spans would be lost. It is estimated that the salvage value of the steel would be about \$0.05/lb if a buyer was available for all the bridge steel, whereas it would be only about \$0.015/lb if it would have to be transported and sold by pieces. The scrap value of the deformed steel would be only slightly less than the latter value or about \$0.01/lb. Thus, unless a buyer for the bridge steel is available, it appears that no significant difference in value of the steel would result. The cost of removal, although not known directly, probably will not be affected by any testing except for the steel salvage value.

In summary, the effect of any anticipated testing program on the costs of bridge removal is expected to be minimal. For the truss bridges, the only effect would be any loss in salvage value of the timber floor system due to destructive testing, and then only if the county desired to remove and salvage it. For the Iowa 89 Bridge, only loads causing permanent deformation in the steel would affect the cost of removal. In this case, even if the entire approach span were deformed and used for scrap, the resulting loss would be about \$12,000. The desire of Boone County to acquire the steel undeformed, although not indicated previously, was considered in developing the test program.

Potential Funding and Cooperation

As noted earlier, several independent agencies or organizations were contacted for possible assistance in funding the research or providing technical assistance. Each organization had been connected in some degree with prior research in related areas. Although believing this research to be significant, most of the agencies were either not involved in funding or had been involved previously to answer questions pertinent at that time. These included the International Road Federation, American Trucking Association, and MTMTS and TACOM of the U.S. Army.

Considerable interest was expressed in providing support by two federal agencies: the U.S. Army Corps of Engineers and Federal Highway Administration Office of Research and Development. Corps of Engineers personnel have been encouraging about cooperating in developing timetables of construction and removal to facilitate testing. The Bridge Division, Office of Research (FHWA) has indicated a willingness to provide substantial instrumentation, test truck and other technical assistance, particularly for research on the beam-and-slab (Iowa 89) bridge. It is recommended that these two agencies be contacted to secure their involvement as the research program develops.

Interest in the research program, particularly on the high-truss bridges, has also been expressed by the offices of County Engineers in the affected counties. Their cooperation in any test program would be essential.

RECOMMENDED TEST PROGRAM

As a result of the feasibility study outlined herein and supported by details provided in Part II, an overall test program has been developed. A general outline of each test is given and the background and expected results for the test are indicated.

Rough cost estimates are provided for general categories of suggested research. Specific proposals will be developed for research as requested. It is felt, however, that this development should await review of this report by the sponsors and potential cooperating agencies.

One critical factor in considering many of the suggested tests is availability of the bridges, which is highly dependent upon the construction and removal time schedule. It may not be possible to determine if time will be available to conduct the tests until shortly before they are initiated.

General Tests on All Bridges

1. Material property tests:

These tests will be made to determine tensile strength, chemical composition, and fracture toughness properties (Charpy V-notch and drop-weight tests).

Although considerable information on material properties has been obtained in recent years, there is a marked lack of data from very old bridges and of toughness data from bridges built about the time of construction of the Iowa 89 Bridge. With the increased concern of the fracture toughness of material and its relation to other material properties, it is recommended that the tests listed be conducted on as wide a variety of material as can be obtained from the bridges. This phase can be coordinated with a general Federal program being conducted in this area.

2. Crack and defect detection:

The Office of Research and Development of the Federal Highway Administration currently has developed ultrasonic flaw detectors, the Acoustic Crack Detector (ACD) and Magnetic Crack Detector (MCD), for use in bridge inspection. The detectors could be used to inspect each bridge prior to removal or testing. After completion of the tests, the inspection results can be compared with actual findings. This program would provide an excellent opportunity to provide system check out.

Tests on Truss Bridges

1. Ultimate load tests.

   (a) Boone County Hubby Bridge — western 165-ft span,

   (b) Dallas County Chestnut Ford Bridge — eastern 150-ft span,

   (c) Polk County Hanley Bridge — eastern 100-ft T3 span.

These bridges were selected because they represent a typical span in each type of bridge found in the area considered, and the ultimate loads computed were low enough to be applied without rock anchors. Bridges (a) and (b) consist of pin-connected trusses supporting timber deck floor systems. The trusses are polygonal-chord and parallel-chord, respectively. Bridge (c) uses standard riveted trusses with a concrete deck floor system. These three bridges represent the major portion of secondary road truss bridges found in Iowa.

The ultimate loads (estimated values shown in Table I-1) will be applied as concentrated loads at midspan (or as a pair of loads at adjacent panel points of midspan if located between panel points). The loads will be applied by hydraulic cylinders tied to a large concrete dead weight poured on river bottomland. All spans selected are over dry land except during high water. Key members in each truss will be instrumented to determine member behavior during loading.

It is recommended that consideration be given to testing the Hubby Bridge span in late Summer 1973. The Chestnut Ford Bridge, if available, could also be tested then or in 1974 in conjunction with the Hanley Bridge tests.

The cost of the initial tests is estimated at $16,500 with subsequent tests at $12,500 each.

2. Load tests on timber floor systems.

The interaction of the timber flooring and the timber stringers is unknown. It is proposed that a series of three tests be conducted on the floor system of the Hubby Bridge (Boone County):

(a) static load tests of in-place floor systems,

(b) laboratory static load tests of timber stringers,

(c) laboratory fatigue tests of timber stringers.

Series (b) and (c) would include stringers representing a spectrum of conditions found in the system. All three series would be conducted on sections of the bridge including regular stringers (western two spans) and creosoted stringers (eastern two spans).

The cost of conducting the static field tests would be about $3,500, if conducted in conjunction with the ultimate load tests of

(a) static,

(b) crawl speed ($\leq$ 5 mph),

(c) dynamic ($>$ 5 mph), and

(d) dynamic with obstruction (plank) on deck.

The basic test program will be conducted using a standard HS20 truck. Indications are that the FHWA will provide its test truck. Some tests will also be conducted using two trucks side by side. In addition, a program of tests (probably with speed restrictions) will be undertaken with a truck simulating a standard HS30 vehicle. This truck, which is 150% of live load design, is well below the HS44 truck, which was calculated to cause first yielding of the main members. These latter tests will provide much needed data for evaluating the effects of increasing legal load limits and overload permits.

It is anticipated that the tests will include:

1. One vehicle loaded to design load in each lane 2 ft from the curb line (or as close as possible to the curb) and also down the center line with the diaphragms still in place.
2. Two vehicles will travel in the center of the bridge side by side with 4 ft between centers of inside wheels.
3. Two vehicles will travel in each lane 2 ft from the curb (or as close as possible to the curb).
4. Test No. 1 will be repeated using an overloaded vehicle.

Instrumentation will include strain gages on the main steel members, on the deck reinforcement, and on the deck concrete. In addition, deflection readings will be taken.

Because of accessibility, testing will be limited primarily to the approach bridge with the instrumentation concentrated in the interior spans. Instrumentation for the study will be provided by the University and augmented for the dynamic studies, based on preliminary indications, by the Federal Highway Administration bridge data acquisition system.

2. Load distribution and deck stresses in bridge with diaphragms removed.

Considerable research has been conducted at service loads on bridges. However, data on the effect of diaphragms on bridge behavior is almost nonexistent. Present criteria[12] for design of diaphragms is based on experience. Since the diaphragms in this bridge are bolted in place, it provides an excellent and unusual opportunity to study their effectiveness and indicate if the present criteria requiring diaphragms is realistic.

The test program conducted will be substantially the same as outlined in subsection (1) above for the as-built bridge.

Assuming that the FHWA is able to provide the test truck and dynamic data acquisition system, the estimated cost for the two previously outlined test programs is $55,000.

3. Fatigue and ultimate strength of in-place deck slab sections.

Only limited field testing has been conducted on the in-place strength of bridge decks and most of this has been conducted on bridges prior to opening. Since this bridge has been in service more than 15 years, the results of the proposed tests will indicate more realistically the actual behavior of bridges in service and, also, the life expectancy of the deck from service loads.

provide an opportunity to obtain information on deterioration not in evidence on the slab surface.

The cost of this phase of the investigation will be about $1,000 if the cores and steel test specimens are provided by another agency, possibly the Iowa State Highway Commission. The cost is primarily for testing the specimens and examining the deck.

5. Ultimate strength of as-built bridge.

Although initially a major thrust of the potential testing program was to study ultimate load capacity of the bridge, the present study has indicated that these tests are neither practical nor feasible. This conclusion is based on:

(a) poor soil condition at the bridge site for anchoring the load mechanism and excessive expense for alternate loading systems. As noted in Table I-2, the ultimate load per wheel (or anchor point) would be at least 105 kips. This magnitude of load would require tension piles as a minimum.

(b) results of Tennessee load tests on a bridge similar to the Iowa 89 Bridge indicating that significant reserve capacity exists in bridges of this type. Thus, it is doubtful if problems in bridge behavior will result from static failure of the structure.

(c) salvage usefulness of the bridge girders. Boone County has expressed a strong desire to obtain the girders for use as a replacement to an existing truss bridge. Any tests of this type would, as a minimum, put substantial permanent distortion in the girders.

Detailed Proposals

The cost estimates provided have been based on the University providing technical services and analysis as well as most test equipment. However, substantial cooperation will also be required from the Corps of Engineers, the Iowa State Highway Commission and the Federal Highway Administration.

Detailed proposals have not been prepared, since a number of potential projects have been outlined. It is recommended that the affected agencies (Corps of Engineers, ISHC, FHWA and County Engineers of Boone, Dallas and Polk Counties) review the projects to determine the areas of specific interest, as well as availability of the bridges for testing. Detailed proposals, including budget estimates, can then be prepared.

Relationship to Federally Coordinated Program

The research outlined herein will fall within the scope of several projects in the Federally Coordinated Program of Research and Development in Highway Transportation (FCP). This program, developed by the Office of Research of the Federal Highway Administration, includes a number of research areas in structures and applied mechanics of interest to the FHWA.

The projects directly related to this investigation are:

- Project VF — New techniques for structural inspection of existing bridges.
- Project VG — Predicting the service life of bridges.

Thus, the results of studies suggested in this report will help in supplying information to a comprehensive program of research of interest to all highway transportation agencies.

Future Programs

During the period of this study, there were several occasions in which notices were given that bridges outside the Saylorville area were scheduled for destruction or considered for removal. It is suggested that an ongoing program be developed in which all bridges in the state considered for removal be studied for possible testing. Depending on the results of the study, these tests could consist of only simple material property tests or could include a much more extensive program.

It is felt that, with the bridge building program of the state and counties, many opportunities to conduct significant research will develop in the future.

PART II—SUPPORTING INFORMATION

SUMMARY AND CONCLUSIONS

As a result of construction of the Saylorville Dam and Reservoir, six highway bridges are scheduled for removal. It is proposed to use these bridges to study the behavior of full-scale bridges. A feasibility study was conducted and the results show that significant information on the behavior of bridges designed for normal service can be obtained from a wide variety of tests.

In summary, it can be concluded from this study that:

(a) for the beam-and-slab bridge:

1. testing to failure is not feasible,
2. dynamic testing at design load and overload levels will provide useful data, and
3. testing of deck components under static and fatigue loads should be conducted.

(b) for the high-truss bridges:

1. ultimate load tests should be conducted on three selected spans,
2. fatigue tests should be undertaken on complete component members selected from all truss bridges, and
3. tests should be conducted on in-place timber decks and timber stringers.

After review of this study is completed and coordination is developed with the affected agencies, detailed proposals for the research can be developed.

CONTENTS

FIGURES

TABLES

INTRODUCTION

As a result of construction of the Saylorville Dam and Reservoir on the Des Moines River, six highway bridges are scheduled for removal. Five of these are old high-truss single-lane bridges, each having several simple spans. The other bridge is a fairly modern (1955) double 4-span continuous beam-and-slab composite highway bridge. The availability of these bridges for overload and destructive tests presents an unusual opportunity for studying the behavior of bridges.

Because of the magnitude of the potential testing program, a contract was awarded to the Engineering Research Institute of Iowa State University by the Iowa State Highway Commission for studying the feasibility of a field investigation of dynamic properties and ultimate load capacity of the bridge superstructures. The results of that feasibility study are presented in this report. This portion (Part II) of the final report includes the supporting detailed information on the study, and Part I summarizes the findings. Part II is intended for use by persons interested primarily in the analytical study of the behavior of the Iowa 89 Bridge. Details are provided on the analysis used to determine the loadings to be used for the suggested tests.

Part I includes the program of study, background in field testing of bridges, description of bridges studied, results of bridge analysis, contacts with other organizations, and the recommended test program. It should be reviewed prior to studying Part II.

SOIL CONDITIONS AT IOWA 89 BRIDGE

One critical consideration in determining if ultimate load tests can be conducted is the soil conditions at the bridge site. Three potential sources of information were examined to see if sufficient anchorage could be obtained.

Initially the soil borings taken by the ISHC prior to the bridge construction were studied. A typical soil boring is shown in Fig. II-1. It can be seen that the subsoil is composed primarily of fine sand and lean clay underlane by gravelly sand and shale and fissle with limestone nodules.

In addition, soil borings taken by the Corps of Engineers at the site of the new railroad bridge about 1000 ft south of the highway bridge and at the new relocated Iowa 89 Bridge were examined. These borings showed material in the subsoil similar to that shown in Fig. II-1.

It can be seen from these data that rock anchors are out of the question. In fact, none of the typical borings reach what could be generously called rock or hard pan. Therefore, the only recourse would be to resort to steel tension piles or cast in-place augered piles. Due to characteristics of the subsoil, an auger-cast pile would probably be the best solution because the concrete is pumped into the augered hole under high pressure, expanding into the soil, and therefore greatly increasing the holding properties of the pile. The cost of such an installation would be about $10,000. It is apparent that, considering the other factors outlined in Part I, the cost of such an installation would not be feasible.

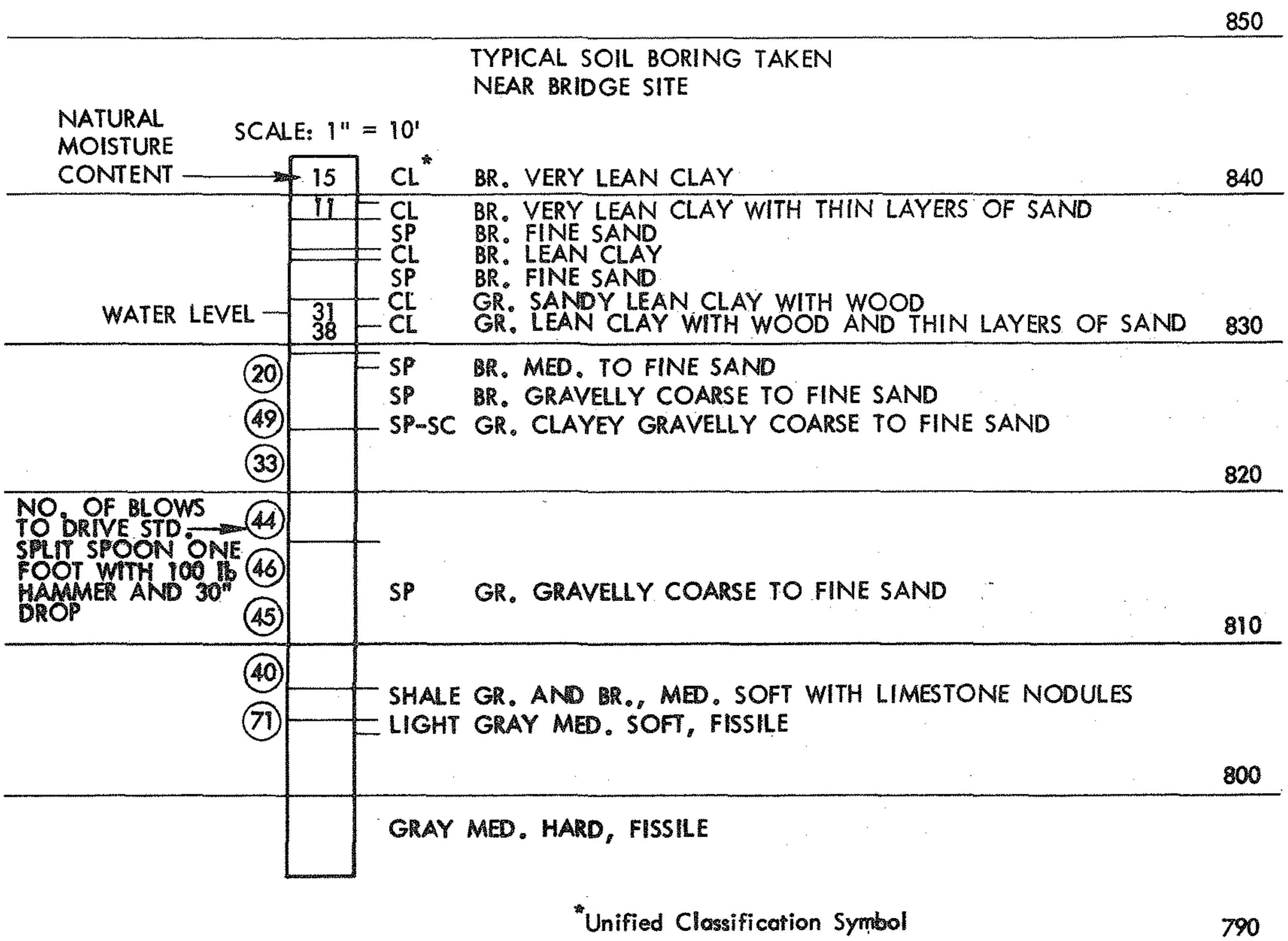

Fig. II-1. Typical soil boring at bridge site.

LONGITUDINAL BEHAVIOR — APPROACH SPAN — IOWA 89 BRIDGE

Bridge Section Properties

Longitudinal and transverse sections for the 306-ft approach span of the bridge are shown in Figs. II-2 and II-3. The assumed section used in determining moments of inertia are shown in Fig. II-4.

The bridge properties were calculated for composite and non-composite action. When the slab is poured, the dead load of the slab, girders, and diaphragms is assumed to be taken by the girders acting alone. Any subsequent loading is assumed to be resisted in the positive moment region by the composite section of girders and slab acting together. The bridge was assumed to be a composite section in the positive moment section, because the shear connectors were designed for this purpose only. The bridge section properties in the longitudinal direction, including the inertia, critical distances, and section moduli for the interior and exterior girders are shown in Tables II-1 and II-2, respectively.

Dead Load Moments and Stresses (86-ft Span)

The first stage of dead loading with noncomposite action between the slab and girders was found using the moment distribution method transversely on the bridge. This was done rather than use a simple-span assumption of support between girders because the exact construction techniques and order of pour used at the site was not known. The simple-span approach would also negate the stiffness effect of the forming used. The second, or composite, stage was found using the same process with the assumed loading on the girders as given in Table II-3.

PART LONGITUDINAL SECTION SHOWING STEEL REINFORCEMENT

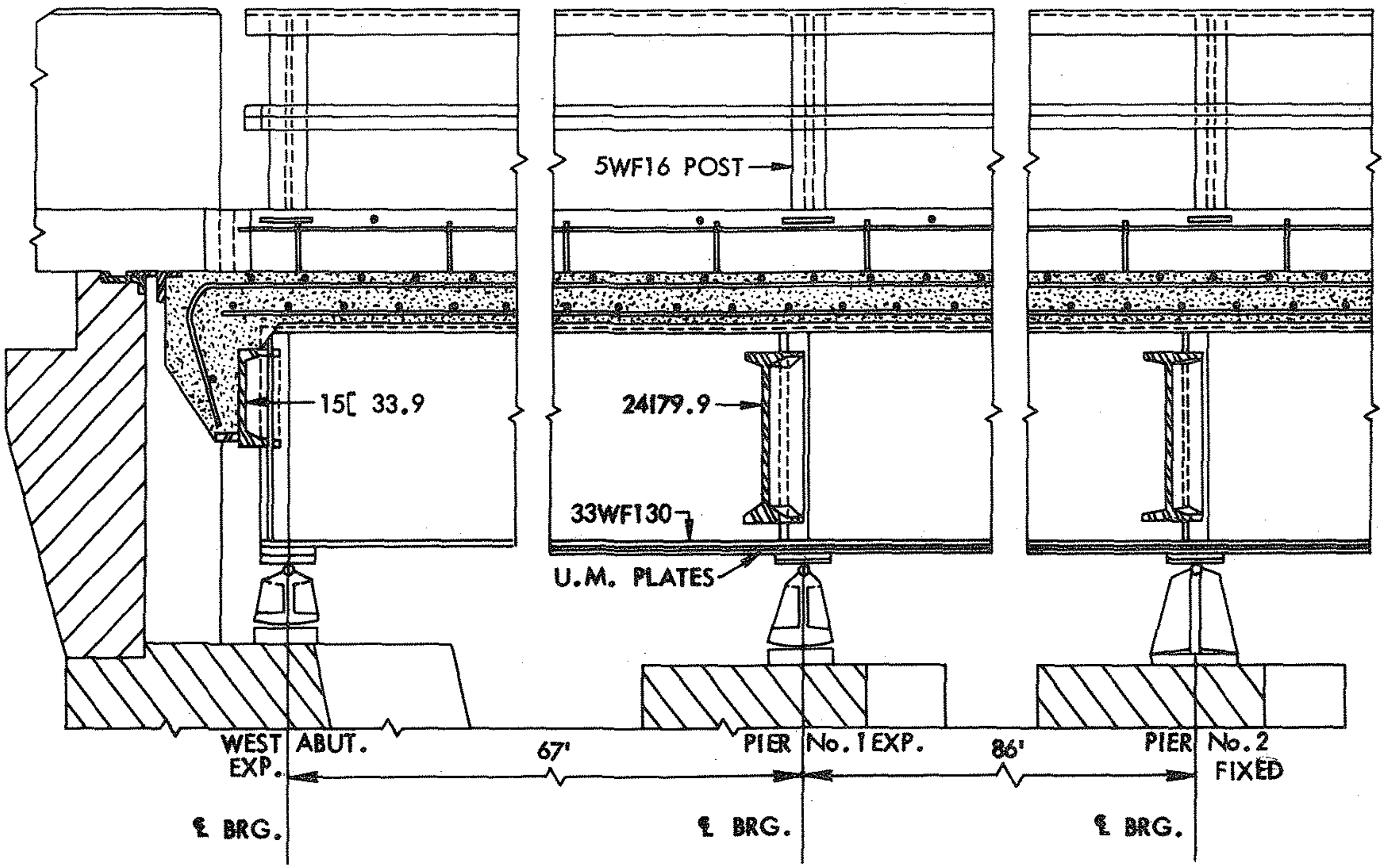

Fig. II-2. Longitudinal section — Iowa 89 Bridge.

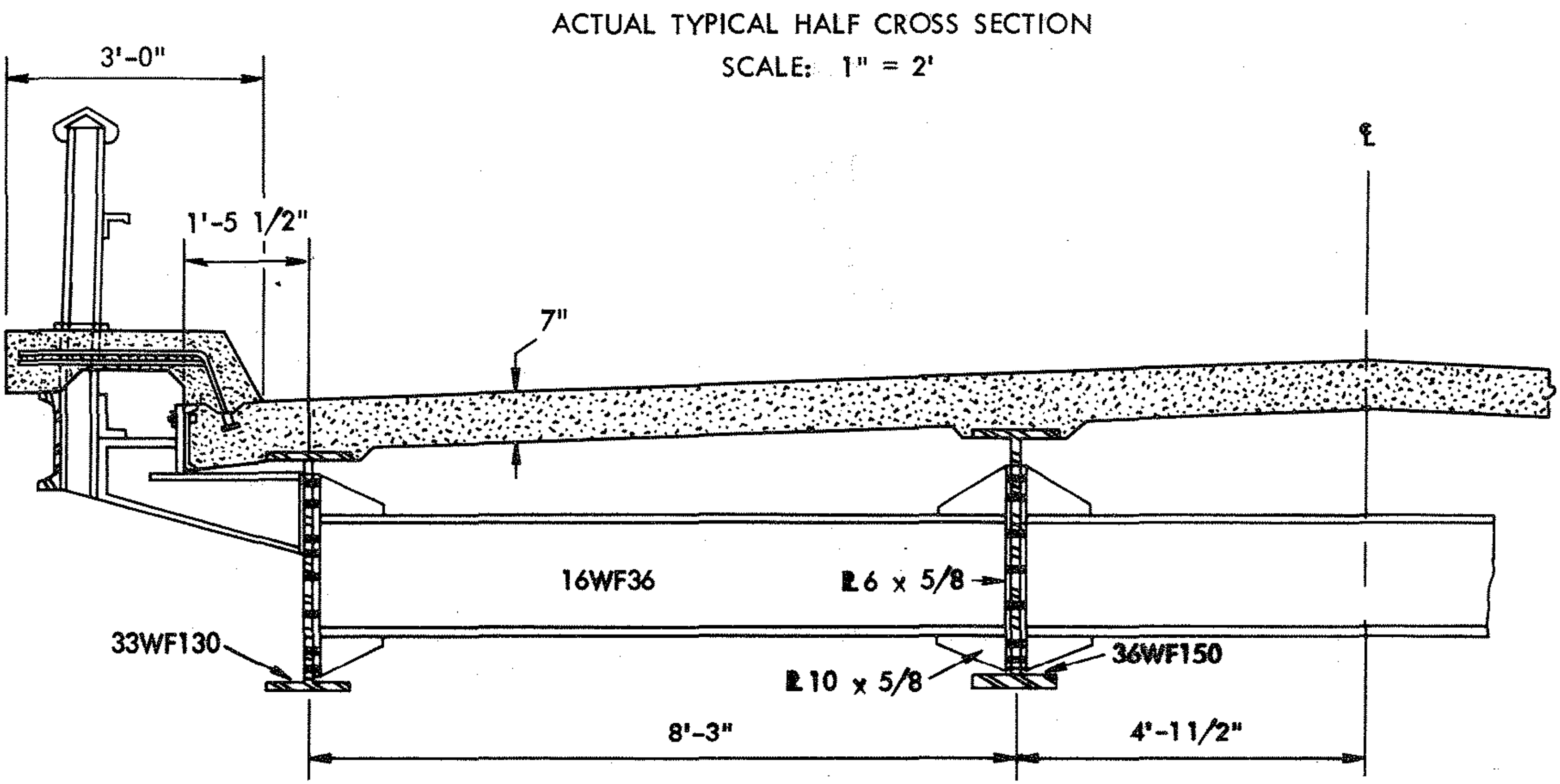

Fig. II-3. Typical cross section — Iowa 89 Bridge.

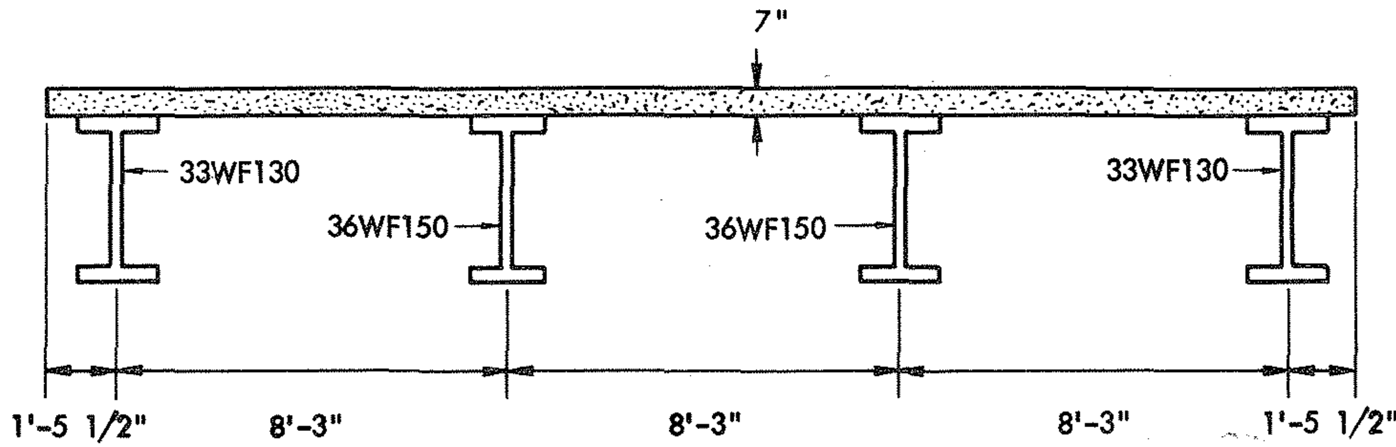

Fig. II-4. Assumed average cross section — Iowa 89 Bridge.

Table II-1. Basic properties for interior girder.

	Positive moment		Piers	
	Noncomposite	Composite	1st CP[a]	2nd CP
N.A. (in.)	—	+ 14.20	—	—
C_{top} (in.)	—	+ 10.72	—	—
C_{stltop} (in.)	+ 17.92	+ 3.72	+ 18.23	+ 18.80
C_{bot} (in.)	- 17.92	- 32.12	- 18.23	- 18.80
I (in.^{4})	9,012	22,795	13,583	17,115
S_{conc} × n (in.^{3})	—	17,011	—	—
S_{stltop} (in.^{3})	502.9	6,127.7	745.1	910.4
S_{stlbot} (in.^{3})	502.9	709.7	745.1	910.4

[a] CP = cover plate.

The moments in the bridge were found using the loadings found in Table II-3 and areas of the influence lines provided by the Iowa State Highway Commission. Figure II-5 shows the total dead load moment in the exterior girder of the bridge. The total dead load moment diagram for the interior girder of the bridge is given in Fig. II-6.

Using the properties of the girder sections, the stresses in the concrete and the extreme fibers of the wide flange section can be found. The dead load stresses in the 86-ft span are shown in Figs. II-7 and II-8 for the exterior and interior sections, respectively.

Table II-2. Basic properties for exterior girder.

	Positive moment		Pier I		Pier II	
	Noncomposite	Composite	1st CP	2nd CP[(a)]	1st CP	2nd CP
N.A. (in.)	—	+ 12.15	—	—	—	—
C_{top} (in.)	—	+ 11.40	—	—	—	—
C_{stltop} (in.)	+ 16.55	+ 4.40	+ 16.86	+ 17.43	+ 16.93	+ 17.61
C_{bot} (in.)	- 16.55	- 28.70	- 16.86	- 17.43	- 16.93	- 17.61
I (in.^{4})	6,699	16,256	10,253	12,986	10,996	14,485
$S_{conc} \times n$ (in.^{3})	—	11,408	—	—	—	—
S_{stltop} (in.^{3})	404.8	3,694.6	608.13	745.3	649.5	822.6
S_{stlbot} (in.^{3})	404.8	566.4	608.13	745.3	649.5	822.6

(a) CP = cover plate.

Table II-3. Dead load distribution to girders (lb/ft of girder).

		Exterior	Interior
Noncomposite			
Slab		430	780
Girder		134	154
Diaphragms		8	21
	Total	572	955
Composite			
Rails, posts, brackets		98	- 26
Curb		345	- 80
	Total	443	- 106

Truck Position for Maximum Moment

In order to maximize the live load moment in the girders for the truck loading it is necessary to determine the truck position on the bridge. Using the ordinates of the influence lines and linear interpolation between points, it was determined that the maximum moment in the 86-ft span was produced with the HS truck facing the 67-ft span with the driving axle placed on the centerline of the span. For determining lateral position of the trucks for maximum loading on a particular girder, it was found that the lane limitations as specified in the AASHO code governed. Thus, for maximum loading on the interior girders, two HS trucks were placed side by side centered on the bridge with the center of the inside wheels 4 ft apart (Fig. II-9). This spacing follows from the use of a 10-ft lane and an assumed spacing between centers of truck wheels of 6 ft. The maximum loading on the exterior

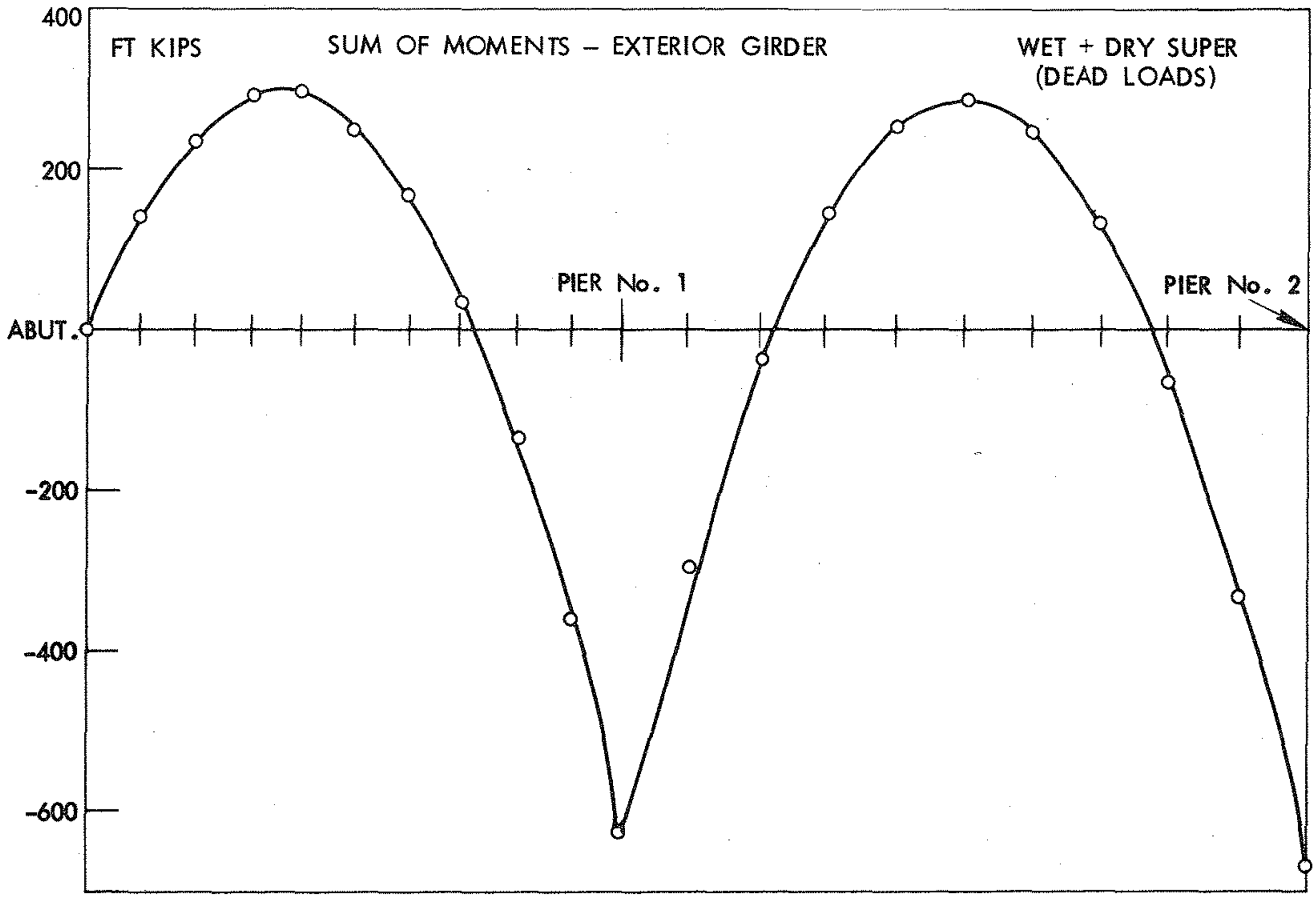

Fig. II-5. Total dead load moment — exterior girders.

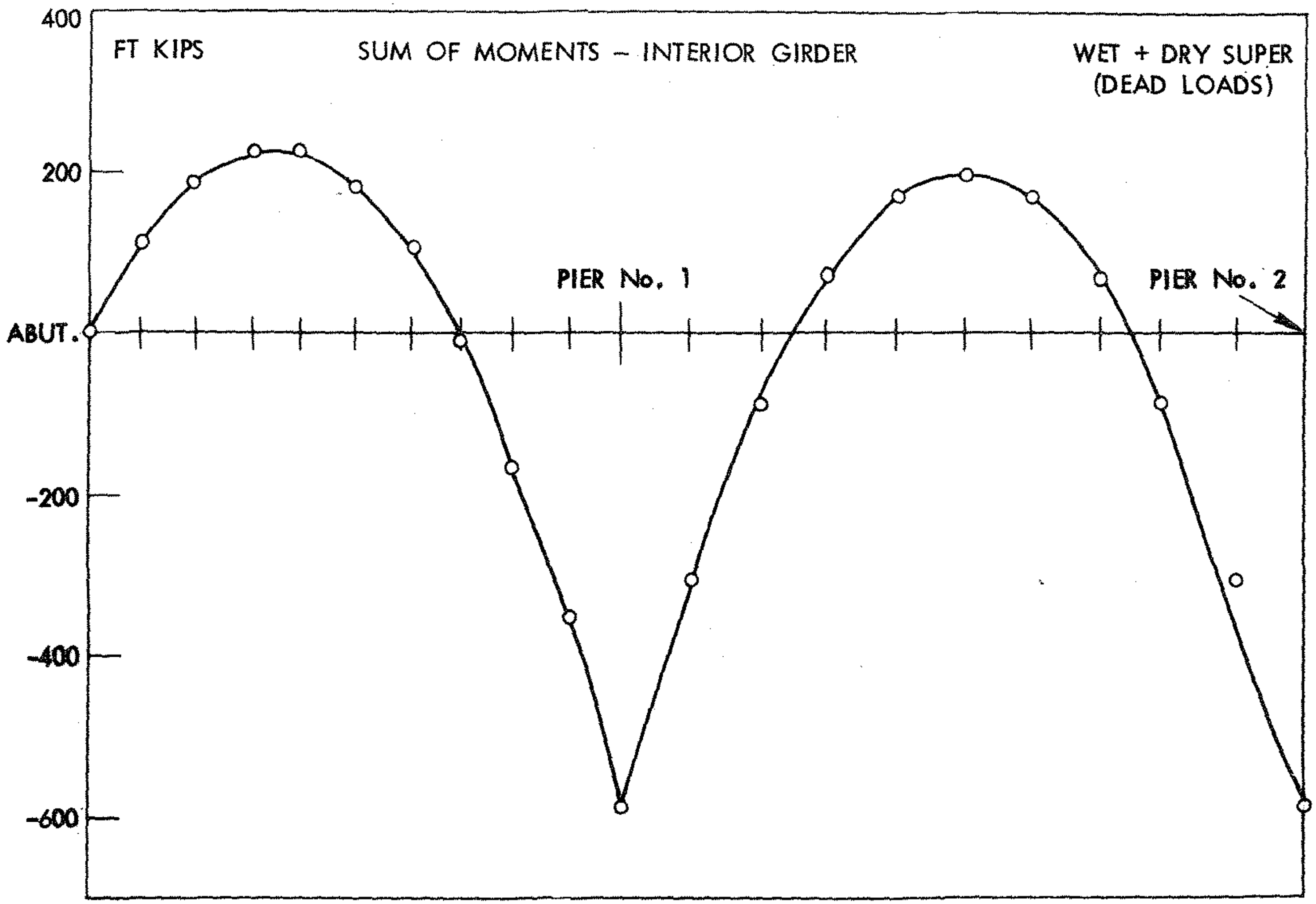

Fig. II-6. Total dead load moment – interior girder.

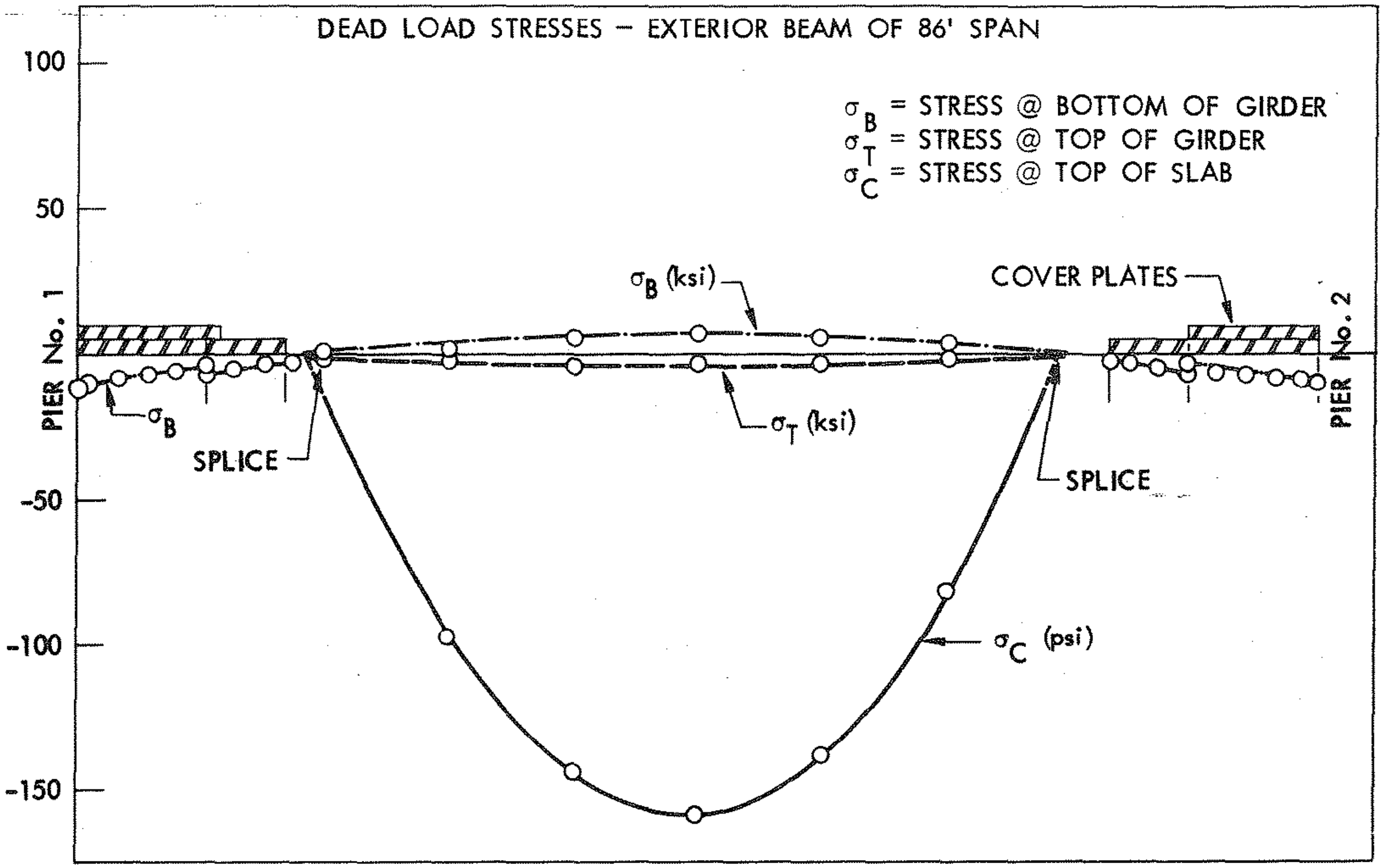

Fig. II-7. Dead load stresses – exterior beam.

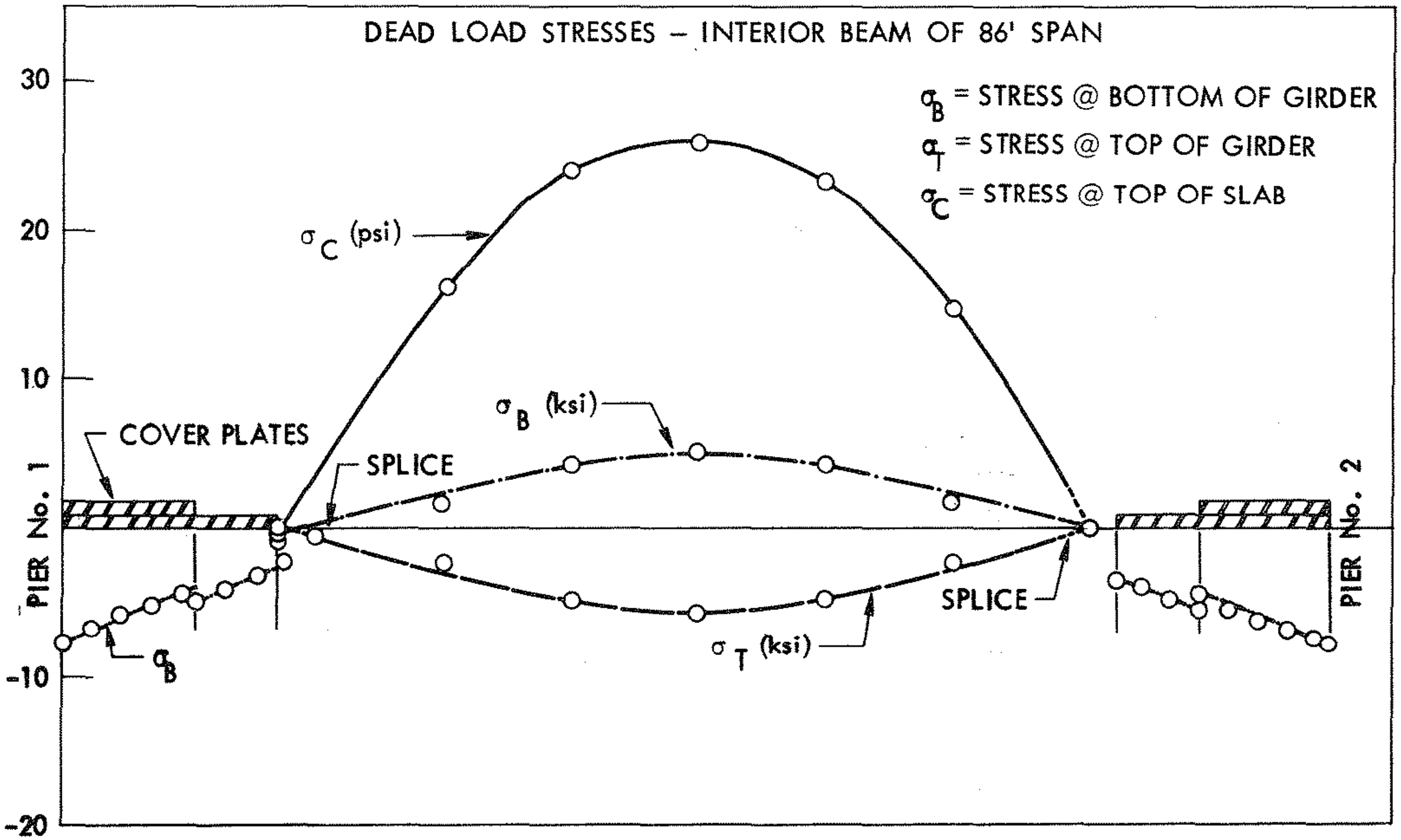

Fig. II-8. Dead load stresses – interior beam.

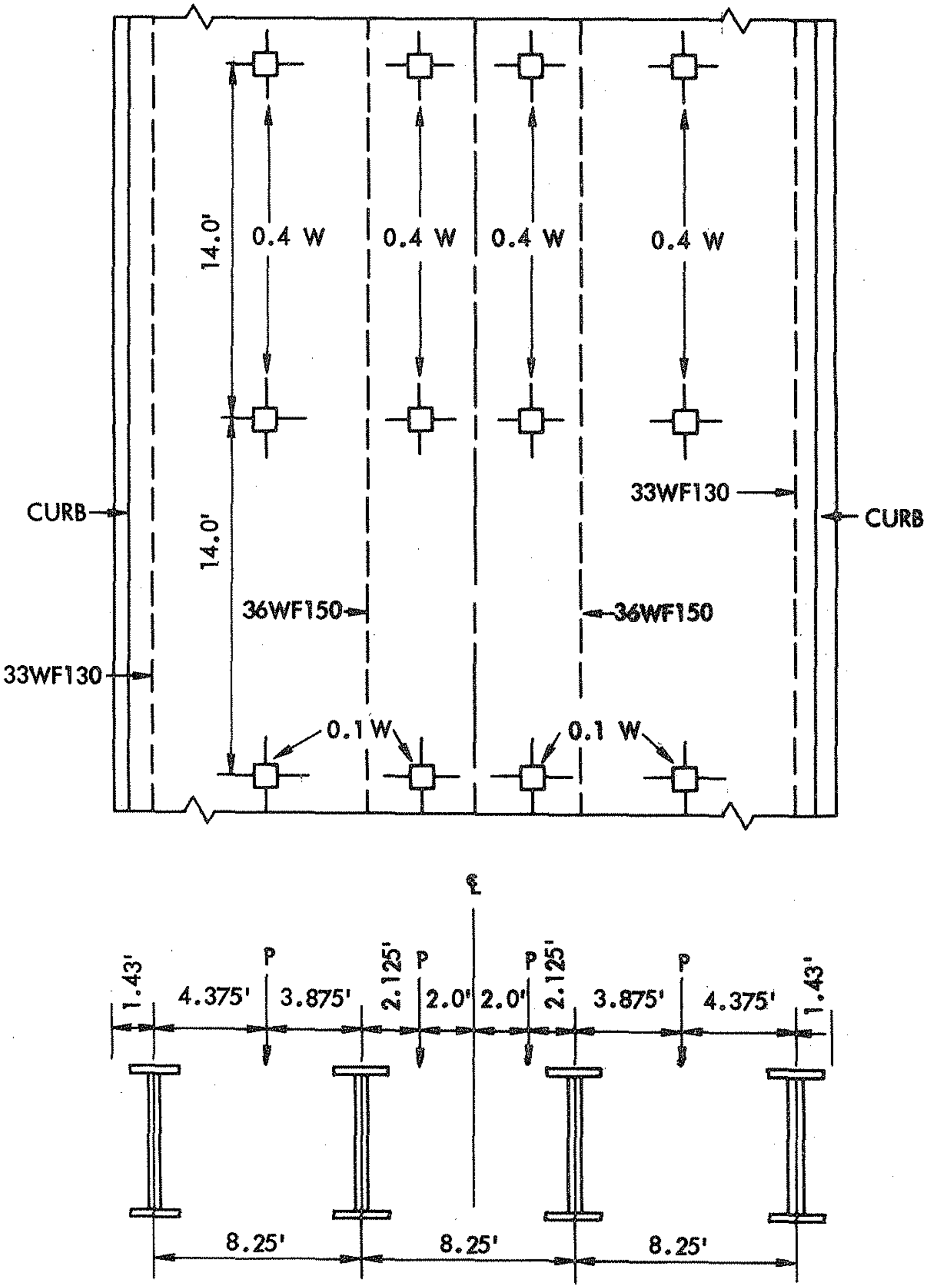

Fig. II-9. Maximum loading position — interior girders.

girders considered two conditions. First, if the slab is assumed to be supported by nonyielding girders, then elastic analysis is applied and two HS trucks are used, each placed with their outside wheels 2 ft from the curb (Fig. II-10). Second, if the bridge is assumed to act as an orthotropic plate, which accounts for yielding of the girders, only one HS truck is required placed 2 ft from the curb.

For the ultimate loading case, the front wheel loads were assumed to be absent and the truck was assumed to be positioned on the bridge such that the midspan of the bridge was centered between the centroid of the truck and an axle. This positioning is the exact placement to produce maximum moment under this axle assuming equal end moments. The lateral placement of the wheels was the same as mentioned above which produced maximum reactions on the interior girders.

Magnitude of Truck Load for Yield of Interior Girders

When two trucks are placed side by side, as shown in Fig. II-9, an elastic analysis of the slab using nonyielding girders results in a load factor of 1.71. In other words, the central girders are assumed to receive 1.71 lines of wheels rather than 1.41 which would be the solution if the slab were assumed to be infinitely stiff and supported by the deflecting girders. If we consider orthotropic plate behavior which considers both aspects of bridge behavior, the load factor becomes 1.96. When 1.96 is used to estimate the load factor, then the moment would equal the magnitude of the wheel loads multiplied by their respective influence line ordinate and the result multiplied by the load factor. Thus, the moment at midspan for a standard HS-20 truck is

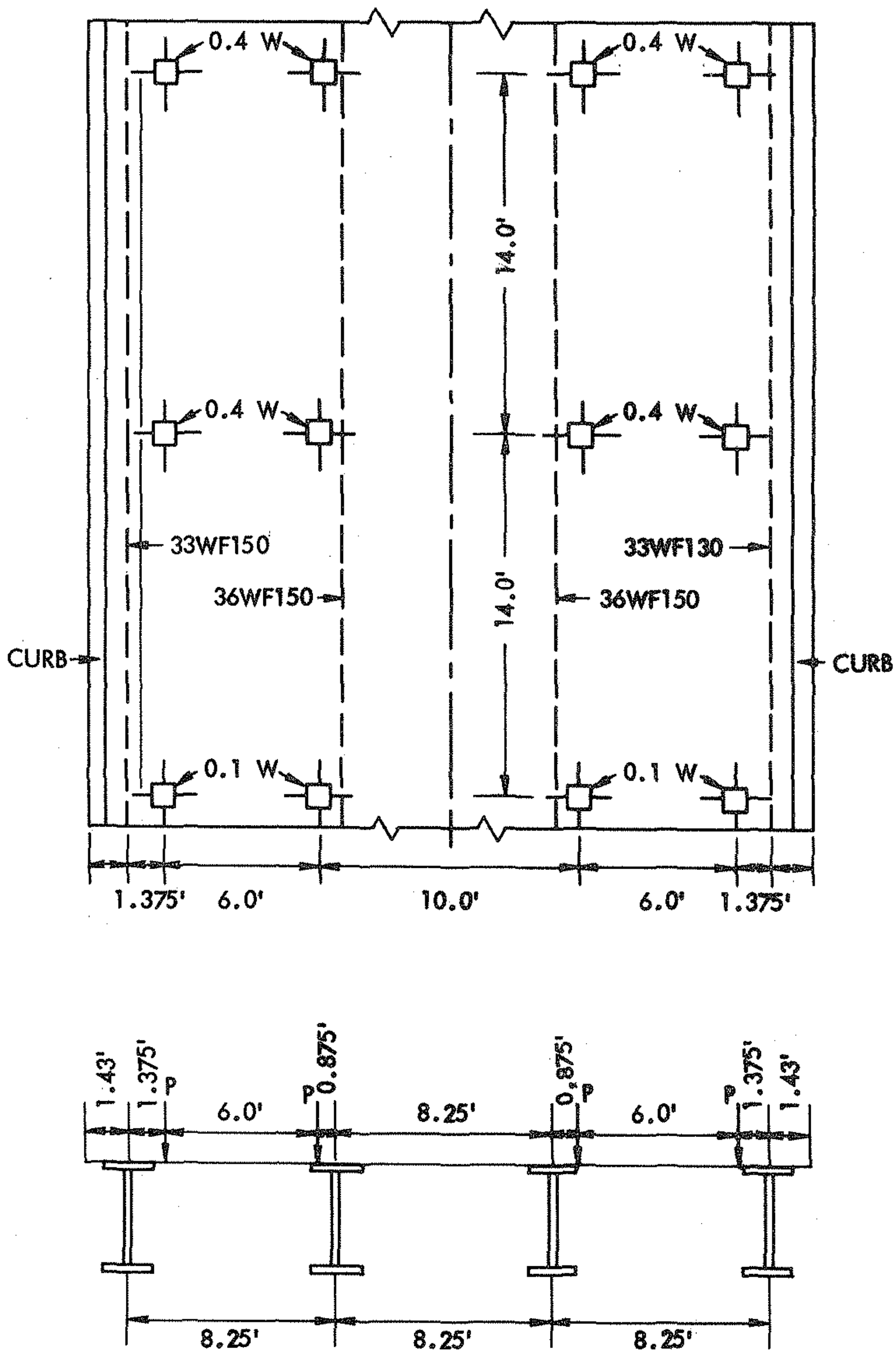

Fig. II-10. Maximum loading position — exterior girders.

$$1.96[(4 \times 8.914) + (16 \times 15.286) + (16 \times 8.951)] = 830.0 \text{ ft-kips.}$$

The dead load stress in the bottom flange at midspan (Fig. II-8) was calculated to be 5.03 ksi. For mild steel with a yield stress of about 36 ksi, a differential stress of 30.97 ksi would be available for live loading before the section yields. Therefore, if we multiply this stress by the section modulus, this will indicate the live load moment which can be applied to the section before yielding occurs. Thus, M_{LL} for yield is

$$30.97 \times 709.7/12 = 1831.6 \text{ ft-kips.}$$

The equivalent HS trucks to produce this loading would be,

$$(1831.6/830.0) \times 20 = 44.1,$$

or an HS-44 with wheel loads of 35.3 kips.

Magnitude of Truck Load for Yield of Exterior Girders

When two trucks are placed on the bridge near the curbs, as shown in Fig. II-10, and an elastic analysis is made assuming nonyielding supports, the load factor is 0.839. When orthotropic plate theory is used, the load factor for the exterior girder is 0.93. Using the value of 0.93, the live load moment in the girder is

$$0.93[(4 \times 8.734) + (16 \times 15.062) + (16 \times 8.752)] = 386.8 \text{ ft-kips.}$$

The dead load stress in the bottom fiber of the exterior girder (Fig. II-7) was calculated to be 7.26 ksi, or 28.73 ksi of live load stress may be added before yielding occurs. Therefore, if we multiply this stress by the modulus, it will yield the live load moment which can be applied. Thus,

$$M_{LL} = 28.73 \times 566.4/12 = 1356.0 \text{ ft-kips.}$$

The equivalent HS truck would be

$$\frac{1356.0}{386.8} \times 20 = 70.1,$$

or an HS-70 with wheel loads of 87 kips.

Magnitude of Truck Loads to Yield Exterior and Interior Girders

When the interior girders yield, the exterior girders pick up more of the applied load. Therefore, it would be reasonable at this stage to conservatively assume that the distribution factors to the exterior girders jump to 0.85 while the interior girders drop to 1.15:

$$\frac{1356.0}{\left(\frac{386.8}{0.93}\right) \times 0.85} \times 20 - 76.7,$$

or HS-77 with a wheel loading of 61 kips.

Truck Loading for Ultimate Moment

In order to determine the ultimate moment capacity of the bridge, the ultimate moment of the composite cross sections must be found. Assuming the concrete strain at ultimate to be 0.003, the resulting moment for the exterior composite beam was computed as 2329.6 ft-kips. The computed moment for the interior composite beam was 2979.9 ft-kips.

These moments were also calculated using approximate methods given in the AISI Bulletin No. 15[2]. These moments were 2342.2 and 2952.6 respectively, which are approximately the same as above. The total resulting positive moment would then be

$$(2 \times 2329.6) + (2 \times 2979.9) = 10{,}619 \text{ ft-kips.}$$

Because the moment diagram for the loaded 86-ft span is very steep in the region of the piers, the final hinges actually form at the ends of the cover plates in the adjoining spans. If the moment is linearized in the 67-ft span and the yield moment is assumed to occur at the end of the cover plate in this span, then the resulting moment over the first pier is computed to be about 8000 ft-kips. The second pier may also be assumed as 8000 ft-kips.

Neglecting the effect of the front wheels (because an anchor system would be used) and using only the eight wheel loads from the main axles, with placements of the wheels located for maximum moment, the resulting single wheel load is 104.69 kips or an equivalent HS-130 truck.

TRANSVERSE BEHAVIOR — APPROACH SPAN, IOWA 89 BRIDGE

Load Distribution

The loading used to determine the fatigue properties and the ultimate load carrying capacity of the slab in the transverse direction were assumed to be a single concentrated load at midspan of the bridge centered on the slab between the exterior and interior girder. Slab stiffness will be based on the assumption that the effective slab width is sixteen times its thickness. This is of course more than can be assumed for load carrying purposes. Figure II-11 illustrates the slab section of the bridge and the system assumed for analysis.

There are two types of solutions for this problem. The first solution is for the elastic case, and the second for the ultimate loading case.

Elastic Loading

The solution for elastic loading is based on the assumption of deflecting girders supporting the slab. If we further assume that the slab section is taken from the midspan of the bridge, then the flexibility of the girders is,

$$F = \frac{L^3}{93.2\ EI}\ .$$

Using L^3/EI as the relative flexibility, for the exterior girders this is,

$$F_1 = \frac{86^3}{16256.1\ E_S} = \frac{39.127}{E_S}\ \mathrm{ft}^3/\mathrm{in.}^2\ \mathrm{kips},$$

and for the interior girder this is

$$F_2 = \frac{86^3}{22795.0\ E_S} = \frac{27.903}{E_S}\ \mathrm{ft}^3/\mathrm{in.}^2\ \mathrm{kips}.$$

TRANSVERSE SECTION PLAN, AND PROFILE SHOWING
LOADING FOR MAXIMUM POSITIVE MOMENT IN
SLAB

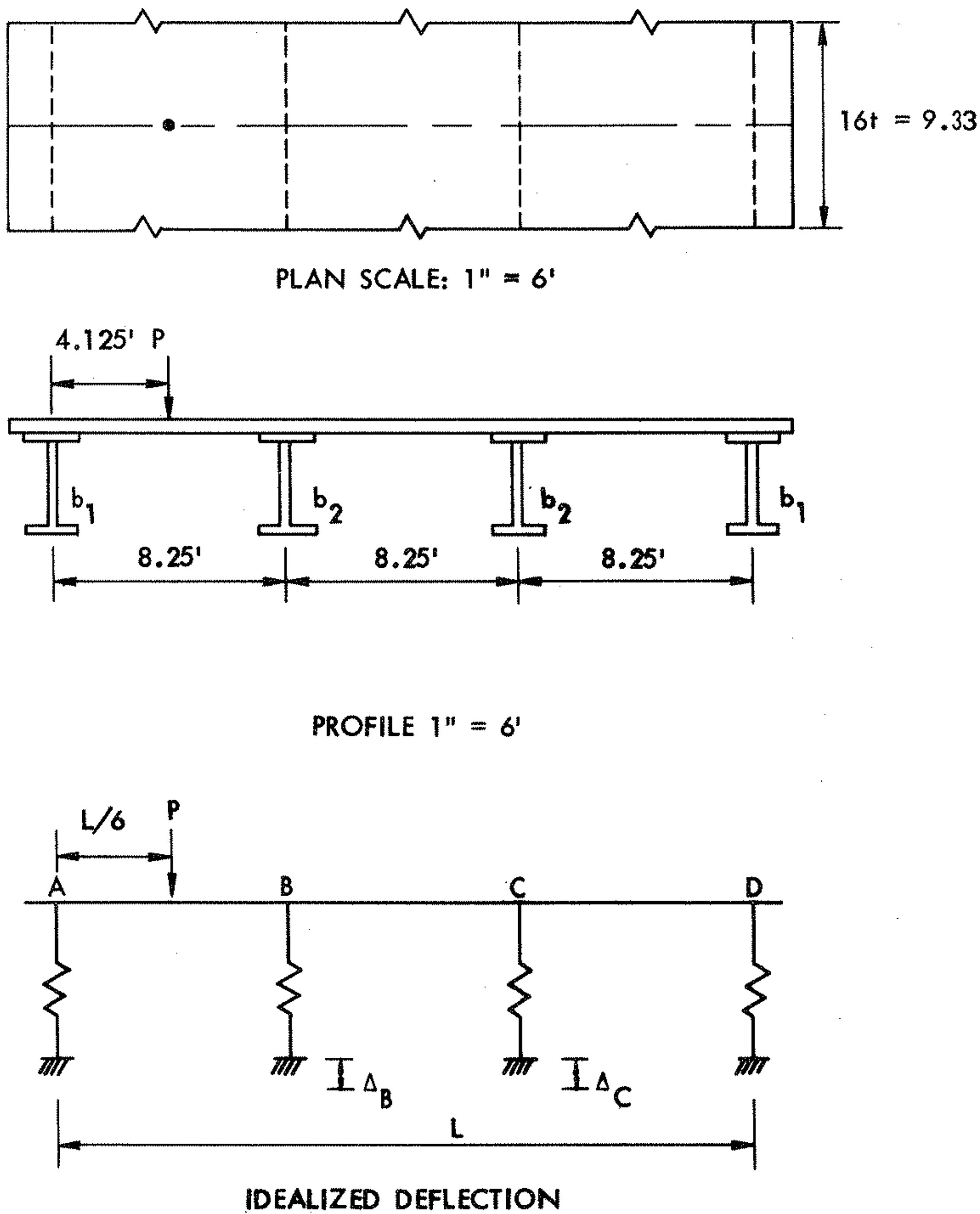

Fig. II-11. Cross section — Iowa 89 Bridge.

$$\delta_{BC} = \delta_{CB} = 0.014403\ F_S + 0.04779\ F_1.$$

Substituting into Eqs. (1) and solving

$$R_B = 0.4797\ P$$

$$R_C = 0.0987\ P$$

gives

$$R_A = 0.4806\ P$$

$$R_D = -\ 0.0590\ P.$$

The maximum moment under the load then is 1.982 P ft-kips. The elastic solution for this moment assuming nondeflecting supports would be 1.65 P. Thus, deflecting of the girders increases the positive moment by 17%. The negative moment over the interior girder is 0.16 P ft-kips, while the nondeflecting girder solution is 0.825 P ft-kips, which is a very large change.

Ultimate Strength

Using the ACI Building Code methods and an assumed steel yield of 50,000 psi and concrete cylinder strength of 4000 psi, the ultimate moment is found as follows:

Transverse Positive

$$d = 4.875\text{ in. } A_s = \#6\ @\ 8 = 0.66\text{ in.}^2/\text{ft}$$

$$a = \frac{0.55 \times 50}{0.85 \times 4 \times 12} = 0.8088\text{ in.}$$

$$M_u = 0.9 \times 50 \times 0.66\ (4.875 - \frac{0.8088}{2})/12$$

$$= 11.06\text{ ft-kips/ft}$$

Transverse Negative

$$d = 5.125\text{ in.}$$

$$M_u = 0.9 \times 50 \times 0.66\ (5.125 - \frac{0.8088}{2})/12$$

$$= 11.68\text{ ft-kips/ft}$$

Longitudinal Positive

$d = 5.125$ in. $A_s = \#6 @ 15 = 0.35$ in.^{2}/ft

$$a = \frac{0.35 \times 50}{0.85 \times 4 \times 12} = 0.429 \text{ in.}$$

$$M_u = 0.9 \times 50 \times 0.35 \ (5.125 - \frac{0.429}{2})/12$$

$$= 6.44 \text{ ft-kips/ft}$$

Longitudinal Negative

$d = 4.875$ in.

$$M_u = 0.9 \times 50 \times 0.35 \ (4.875 - \frac{0.429}{2})/12$$

$$= 6.12 \text{ ft-kips/ft}$$

Lower Bound for First Yield

The maximum positive moment in the slab varies from zero at some distance from the load to a maximum value beneath the load. Using plate theory on a simply supported slab, the equivalent simple beam width to produce the same maximum moment is in this case about 5 ft. Thus, a lower bound value to produce yielding in the slab under the load would be

$$P = \frac{11.06 \times 5}{1.982} = 27.9 \text{ kips.}$$

Ultimate Load Capacity of Slab

Due to the load carrying capacity of the slab in both the transverse and longitudinal directions, yield line theory will provide the best results in computing the ultimate load. Yield line theory uses the principle of virtual work applied to the failure mechanism of the slab. This method simply sets the total external work done by the mechanism equal to the total internal work done in the slab when the mechanism goes through a virtual displacement.

If the unit load method is used to determine the solution, and the reactions at B and C are used as the redundants, the relative flexibility of the slab in terms of length and EI is

$$L^3/E_C I = \frac{24.75^3 \times 12}{E_C \times 16 \times 7^4} = \frac{4.736}{E_C} \text{ ft}^3/\text{in.}^2 \text{ kips.}$$

Using $E_S/E_C = 8$ then,

$$F_S = L^3/E_S I = 4.736 \times 8 = \frac{37.887}{E_S} \text{ ft}^3/\text{in.}^2 \text{ kips.}$$

The relationship between the transverse and longitudinal relative flexibilities is of the same order of magnitude and will have a definite influence on the solution.

The compatibility equations for the problem are

$$\begin{aligned} \Delta_B &= 0 = \Delta_{BO} + R_B\delta_{BB} + R_C\delta_{BC} \\ \Delta_C &= 0 = \Delta_{CO} + R_B\delta_{CB} + R_C\delta_{CC} \end{aligned} \tag{1}$$

where

Δ_{BO} = deflection at B with no restraint at B or C,

Δ_{CO} = deflection at C with no restraint at B or C,

δ_{BB} = deflection at B for unit reaction at B,

δ_{CC} = deflection at C for unit reaction at C,

δ_{BC} = deflection at B for unit reaction at C,

δ_{CB} = deflection at C for unit reaction at B.

Substituting in terms of load and flexibility

$$\Delta_{BO} = 0.09774 \text{ PF}_S + 0.006571 \text{ PF}_1,$$

$$\Delta_{CO} = 0.007973 \text{ PF}_S + 0.004182 \text{ PF}_1,$$

$$\delta_{BB} = \delta_{CC} = 0.016461 \text{ F}_S + 0.005974 \text{ F}_1 + 0.010753 \text{ F}_2,$$

If we assume that the load P at the center of the plate moves through a unit deflection, then the ultimate moments on the transverse line would rotate through an angle of 1/d, the ultimate moments on the longitudinal lines would rotate through an angle of 1/4.125, and the diagonal lines would rotate through an angle of 1/c. Thus,

$$\text{External Work} = P \times 1$$

$$\text{Internal Work} = (M_{u_{AB}} \times 2/d) + (M_{u_{BC}}/4.125) + (M_{u_{AC}} \times 2/c)$$

where

$$M_{u_{AB}} = 6.12 \times 8.25 = 50.49 \text{ ft-kips},$$

$$M_{u_{BC}} = 11.68 \times 2 \times d - 23.36 \times d \text{ ft-kips},$$

$$M_{u_{AC}} = 2 \times S \times (8.75 + 2.31 \cos 2\theta) \text{ ft-kips}.$$

The term $M_{u_{AC}}$ was formulated on the basis of a Mohr's circle construction varying from 11.06 ft-kips/ft for theta equal zero to 6.44 ft-kips/ft for theta equal 90^o.

The solution for this problem is found by minimizing the value of P. This is accomplished by successive trials of d. The optimum value of d for a minimum value of P was found to be 2.595 ft. The corresponding value of P was found to be 73.11 ft-kips.

The ultimate load capacity of the slab based on shear for a 12-in. diameter plate using the ACI Code is

$$P = 4 \times \frac{4000}{1000} \times 0.85 \times (4.875 + 12) \times 4.875$$
$$= 55.65 \text{ ft-kips}.$$

Thus, actually a 16.3-in. diameter bearing plate would be required as a minimum to support the ultimate concentrated load.

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