FINAL REPORT

UTILIZATION OF STEEL IN INDUSTRIALIZED HIGHWAY BRIDGE SYSTEMS

by

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SUMMARY

The space frame concept presented in this report represents the results of an effort to minimize on-site construction time while utilizing steel to provide a high quality but competitive type of bridge structure. A necessary part of the effort was the development of a concept which minimizes material and labor requirements but allows for the systematic coordination of fabrication, transportation and construction. The final evaluation of the space frame concept is enhanced by the cost estimates and comments received from qualified bridge fabricators and contractors in Virginia.

Figures 12 and 13 show the basic relationship between the estimates received from the fabricators and contractors who were supplied with the plans for a representative prototype space frame and three conventional alternatives. The estimates for the prototype structure can be considered as representative of the space frame concept based on the appropriateness of Plan D (Figure 4) and on the current facilities and levels of technology of the companies providing the estimates. A summary of the relationship between the estimates and the basic features of the space frame concept as represented by Plan D are as follows:

> The axially loaded members of the space frame provide a rigid structure with a minimal amount of steel. (The efficient use of steel in a bridge structure tends to reduce the overall material costs and over the years to conserve steel supplies.) Although the current cost per pound of steel is greater for tubular material than plate material, the significant reduction in weight provided by the space frame geometry tends to offset the higher unit cost.
>
> The 6" (15 cm) thick precast deck of the space frame is possible because of the 6' (1.83 m) spacing of the longitudinal steel tubes. Although the 6" (15 cm) deck is 25% thinner than the 8" (20 cm) deck in the conventional plans, the anticipated cost savings are somewhat offset by the current high relative unit cost of precast concrete. Nevertheless, substructure, transportation and handling costs are reduced by the lightweight nature of the preassembled space frame units. Additional savings can be expected as span length increases. Deck maintenance costs should also be reduced because high quality concrete can be obtained with precast construction.
>
> The repetitive geometry of the steel members of the space frame accommodates fabrication. For example, the prototype structure consists of 8" x 6" (20 cm x 15 cm) rectangular tubes, 3" (7.6 cm) nominal diameter pipe, 6" x 4" (15 cm x 10 cm) rectangular tubes, and 6" (15 cm) channels. With the exception of members over the support, similar types of members are the same length and are cut on the same angle. Repetition in fabrication is further accommodated by specifying that camber

will be provided during the erection of the space frame sections by placing steel plates between the appropriate hubs. Nevertheless, a diversity of facilities for and familiarity with space frame fabrication is apparent from the estimates shown in Figures 12 and 13.

The standard 12' (3.66 m) length (Figures 10 and 11) of the space frame provides for convenience and maximum hauling economy. Although transportation costs amount to only a small percentage of the total bridge costs, transportation can play a significant role in the acceptance of an industrialized concept by the bridge building industry.

The bolted type of field connection utilized in the space frame should provide for minimal on-site construction time and costs. Although some apprehension is apparent from the diversity of estimates for the construction of Plan D, it is anticipated that with experience the space frame can be erected quickly and easily and more economically than is suggested by the lowest estimate.

For several reasons it is recommended that precast parapets be considered as an additional feature of the space frame. First, the estimates indicate that cast-in-place parapets cost more for Plan D than for the conventional plans because of the unconventional nature of the space frame construction. Precast parapets should eliminate this added cost and also eliminate the possibility of damaging protruding reinforcing bars during the transportation and handling of the space frame sections. Reduced on-site construction time would be an additional advantage of precast parapets.

The basic features offered by the preassembled space frame represent an effort to efficiently utilize steel in industrialized bridge construction. The estimates indicate that the prototype structure can be fabricated, transported, and erected at a cost which is competitive with that of the three conventional alternatives. The future of the preassembled space frame will depend on the success of field implementations and the significance of the economic benefits that can be derived from material savings, in-house fabrication, and reduced on-site construction time.

PREFACE

In June 1972, I began a trainee program under the supervision of Harry E. Brown, who heads the Industrialized Construction Section at the Virginia Highway and Transportation Research Council. At that time numerous materials were under investigation for use in industrialized bridge construction. Precast concrete was being studied in several projects by Mr. Brown. The merits of plastic were being investigated by Dr. F. C. McCormick of the University of Virginia. Timber bridge research was also under consideration. To complete the list of potential materials under study, I was assigned to investigate the utilization of steel in industrialized bridge construction. Dr. David Morris of the University of Virginia was appointed on an one-eighth time basis to provide technical guidance throughout the project. This report is the result of our research of steel.

The project began with an informal literature survey of industrialized bridge construction with steel. Upon completion of this survey the space frame concept was selected as the most worthy candidate for an in-depth study. A working plan was written and the project proceeded with the development of an industrialized bridge system using available steel components in a space frame geometry. A Fortran computer program was written for the preliminary design of the space frame. The final system was compared on a quantitative basis with a three-span continuous bridge, a structure with three simple spans, and a rigid frame. Cost figures for each of the four structures were based on April 1974 estimates, which were obtained through the courtesy of several steel fabricators and bridge contractors in Virginia.

Although most of this report is original much of it involves the experience and work of others. In particular, Dr. Morris is responsible for many of the concepts and ideas that were developed. In addition, he designed the two bridge substructures and provided the basic plan drawings shown in the report. Both Dr. Morris and Mr. Brown provided guidance throughout the project.

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INTRODUCTION

Traditional bridge construction practices are generally plagued by excessive site construction time, relatively poor and unsafe working conditions, and high labor costs. Industrialized procedures can reduce these problems by concentrating most of the construction effort in the factory rather than the field.

Numerous efforts have been made to industrialize the construction of bridges consisting of wood, concrete, steel and various combinations of these materials. For example, the American Institute of Timber Construction has developed a glulam stringer and deck system that meets many of the requirements for industrialized bridge construction.(1) Similarly, various types of precast concrete members such as longitudinal and segmental box girders, various types of beams, slabs and combinations of beams and slabs have been developed.(2) Steel stringers have been used with the glulam deck system and with the concrete slabs. For example, Purdue University has conducted a study of the construction technique consisting of placing 4' (1.22 m) long by 6" (15 cm) thick full-width transverse concrete panels on steel stringers.(3) Likewise, the United States Steel Corporation has developed a systems technique whereby concrete panels may be placed in either the transverse or longitudinal direction.(4) Finally, a composite industrialized steel and concrete bridge section may be formed by precasting a concrete slab onto an inverted steel T-section.(5)

Industrialized bridge construction as described by these examples involves the field connection of precast, preassembled or modular components. Steel stringers, reinforcing bars, cables, and various steel connections are used in many of these techniques. The same steel products are also required for many forms of conventional bridge construction. To appropriately distinguish between the utilization of steel in industrialized as opposed to conventional bridge construction, the former was defined for the purpose of this report as the use of preassembled steel units designed to minimize on-site erection time.

One example of the successful use of a prefabricated metal bridge is the fast-assembly bridge erected over the Aegidientorplatz in Hannover,Germany.[6] The bridge was assembled from bolted, interchangeable, variable-width, sectionalized components capable of spanning various lengths and variable curves. In 1964, a similar prototype modular steel deck bridge was installed near Baltimore, Maryland, by the Bethlehem Steel Corporation.[7] Also, the N & W Railroad erected a similar bridge over Rt. 340 near Front Royal, Virginia, in 1973.[8] In each of these three bridges, all-steel longitudinal modular units were used. The basic disadvantages of this type construction are (1) the high cost of a short span bridge with an all-steel as compared to all-concrete superstructure, and (2) the span length limitation of longitudinal type construction.

Longitudinal single-cell steel box girder sections were used for each lane of the I-480 bridge erected in Omaha, Nebraska.[9] Although the steel sections provided for quick erection the traditional cast-in-place deck required the usual site construction time.

The accomplishments with quick-assembly steel bridges have clearly been limited. In addition, the structures mentioned have all consisted of longitudinal construction. The author was unable to find work related to quick-assembly transverse steel segmental bridge construction. Relative to concrete, steel offers the advantages of reduced weight and ease of handling. These advantages, along with recent developments in numerically controlled fabrication methods,[10] suggest that steel should be given greater consideration as a material for industrialized bridge construction.

The building industry has experienced a recent success with steel truss construction that has made steel more competitive with other materials.[11] Although this success has been limited to the building industry, the space truss concept plays a role in the transportation industry in the form of aluminum alloys. For example, aluminum space frames have been successfully used for the bridge span of numerous sign support structures.[12] In addition, the U. S. Marine Corps is currently developing an aluminum alloy Sectionalized Assault Bridge.[13] The structure consists of an upper and lower longitudinal truss web extrusion flange which is connected to intermediate tubular members by welded hemispherical segment connections. These recent developments suggested a need for a study of the feasibility of the space frame for bridge construction.

OBJECTIVE

The objective of this project was to develop the concept of an industrialized, preassembled bridge system using steel components presently available in the market place. The following characteristics were considered desirable for the system.

(1) Safety and service in the structure

(2) Efficiency in fabrication and transportation

(3) Minimal erection time and inconvenience in the surrounding area

(4) Minimal cost

(5) Minimal maintenance

(6) Aesthetically pleasing appearance

SCOPE

A review of literature related to the use of steel in industrialized bridge construction reveals the broad nature of the title of this report. A more appropriate title would have been "preassembled steel space frame bridge construction," since the emphasis of this project was placed on an in-depth study of the unresearched area of transverse steel space frame segmental construction for bridge superstructures. The actual work centered on a case study of a quick-assembly steel space frame with a precast concrete deck. An optional precast shell pier assembly with a posttensioned cap beam provided an additional innovation.

The space frame was selected for study for the following reasons:

(1) Material savings, because of the inherently rigid and efficient nature of a space frame section with a truss configuration.

(2) Quick assembly because of reduced weight, ease of handling, and repetitive sections.

(3) Reduction in fabrication problems because of the availability of modern numerically controlled equipment.

(4) Recent success of similar designs in the building industry.

(5) Little research has been done to date on the use of the steel space frame for industrialized bridge construction.

To provide some basis for judging the suitability of the space frame, it was believed that a quantitative comparison with different types of conventional bridge construction was necessary. Consequently, the rigid frame bridge at the intersection of I-64 and Rt. 250, approximately two miles (3.2 km) east of Charlottesville, Virginia, was selected to provide readily available data on one example of conventional bridge construction and to establish a site condition for which a structure with three simple spans, a continuous steel bridge, and an experimental space frame were designed.

Although there may be some limitation in a comparative study using a particular site condition, it was believed that the feasibility of the overall concept of industrialized steel space frame superstructures could be given adequate consideration. Results applicable to other site conditions can be implied, with proper judgement and adequate adjustments, for the particular site used in the case study. In addition to providing an indication of the feasibility of the space frame, a secondary result of this project is a quantitative comparison of three conventional types of bridge construction.

DESIGN

Where applicable the four structures presented in this report were designed for allowable stresses based upon the following:

(1) American Association of State Highway Officials Standard Specifications for Highway Bridges, 1973.

(2) Virginia Road and Bridge Specifications, 1966.

Based on these two references, the following general notes also apply.

(1) Structural steel conforms to ASTM A36 except where noted

(2) Welding conforms to AASHTO M183 specifications

(3) All field connections are ASTM A325 high strength bolts

(4). Pile caps and abutments consist of Class A3 concrete

(5) Decks, parapets and piers consist of Class A4 concrete

(6) Precast shells conform to ACI specifications

(7) Precast shell and fill concrete is Class A5

(8) Main reinforcement conforms to ASTM A615-60 grade steel;

ties and stirrups are 40 grade steel

(9) Piles conform to ASTM A36 steel except as noted and are driven by a drop hammer developing not less than 15,000 foot-pounds (20,300 joules) of energy

(10) Piles are driven to refusal as defined by no more than 1/4" (0.64 cm) penetration in 5 blows

Since each of the four bridges is designed for the same site conditions, the following similarities can be noted.

(1) Capacity is based on HS 20-44 loading and 500,000 stress cycles

(2) Roadway clearance is 16' - 9" (5.11 m)

(3) Parapets and aluminum rails provide 512 lbs./ft. (7.47 kN/m) dead load per lane

(4) Sidewalks are not required

(5) Total length is 214' - 8" (65.43 m)

(6) Number spans is 3

(7) Center span length is 130' - 4" (39.72 m) (comparable for rigid frame)

(8) End span length is 42' - 2" (12.85 m) (comparable for rigid frame)

(9) Total width is 33' - 4" (10.16 m)

(10) Roadway width is 30' (9.14 m)

(11) Number of design traffic lanes is 2

(12) Width of design traffic lane is 15' (4.57 m)

(13) For the conventional Plans A, B, and C (Figures 1, 2 and 3)

(a) Girder spacing is 9' - 2" (2.80 m)

(b) Distribution factor is $\frac{9.17}{5.5 \times 2} = 0.834$ lane

(c) Uniform lane load is $0.640 \times 0.834 = 0.534$ kips per ft. (7.79 kN/m)

(d) Concentrated lane load is $18 \times 0.834 = 15.01$ kips (66.77 kN) (moment)

(e) Concentrated lane load is $26 \times 0.834 = 21.68$ kips (96.44 kN) (shear)

(f) Truck load is 32 x 0.834 = 26.65 kips (118.6 kN)
and 8 x 0.834 = 6.66 kips (29.6 kN)

(g) Deck thickness is 8" (20 cm) (dead load = 917 lbs./ft. (13.4 kN/m) per girder)

These similarities reduced calculation time and produced results for the given site conditions that reflect the relative costs of each of the types of construction included in this project. Approach slabs and the additional height of a superstructure as it affected the approach elevation of the roadway were not considered in the results.

A summary of the design information, final detail drawings, and related discussion for each of the four structures studied follows.

Plan A — Rigid Frame

The design for Plan A was lifted from the plans for the rigid frame bridge at the intersection of Rt. 250 and I-64 approximately two miles east of Charlottesville, Virginia. The actual structure, designed by Hayes, Seay, Mattern & Mattern, of Roanoke, Virginia, consists of five rigid frame girders that cross Rt. 250 at a skew. For the scope of this project a 30' (9.14 m) roadway requiring only four girders and a 90° crossing was considered more appropriate. Nevertheless, the existing rigid frame girder design could be applied to the site conditions and therefore, for convenience, design quantities were taken from work sheets and final design drawings for the existing bridge.

The basic design information was given above and the final design details are shown in Figure 1. The controlling moments and shears and the impact factors for the rigid frame are shown in Figures A-1 and A-2 of Appendix A.

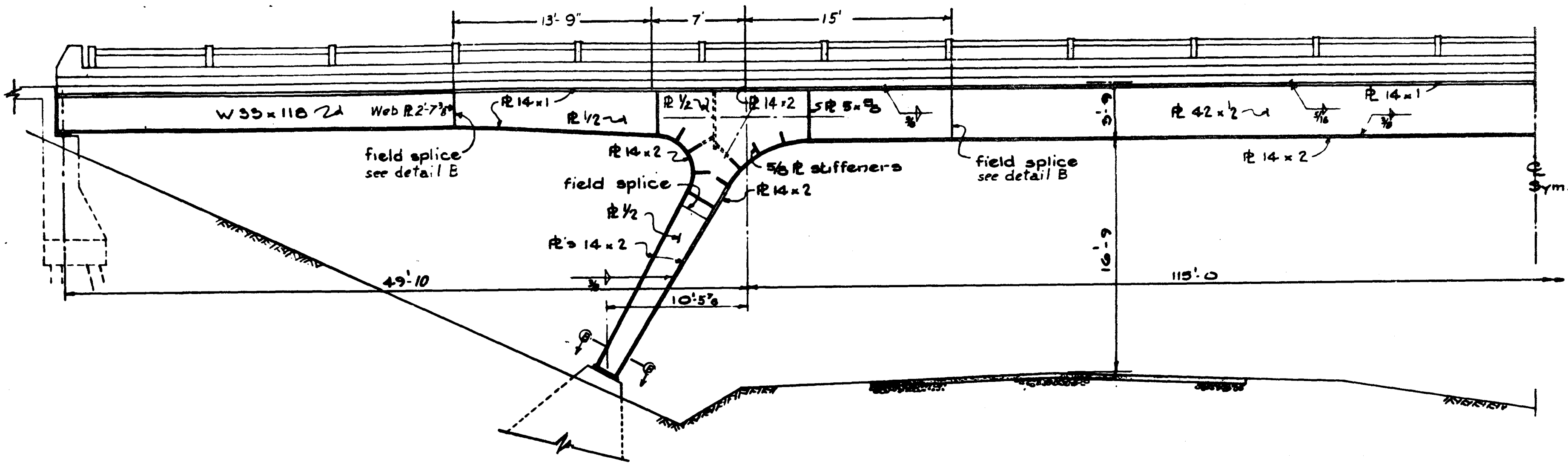
Plan B — Simple Span

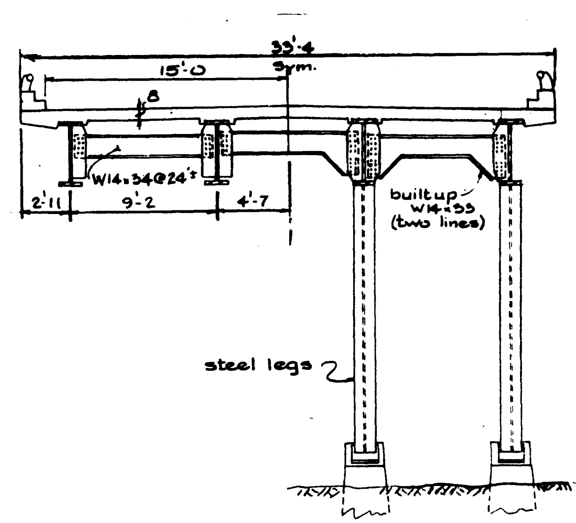
While the rigid frame design was lifted from the plan details of an existing structure, Plans B, C, and D are completely original. Plan B represents an effort to provide an economical simple span design for the given site conditions.

The basic design information is given on pages 4, 5 and 6 and the final design details are shown in Figure 2. The controlling moments and shears and the impact factors for Plan B are shown in Figures A-3 and A-4 of Appendix A.

Plan C — Continuous Span

Moment distribution was used to determine an approximate size for a continuous steel girder. This design was later refined by trial and error with the aid of a canned computer program. Input data consisted of trial beam properties and AASHTO loadings. Output consisted of slope, deflection, moment, and shear. The controlling design quantities are shown in Figures A-5 and A-6 of Appendix A and the final design details are shown in Figure 3.



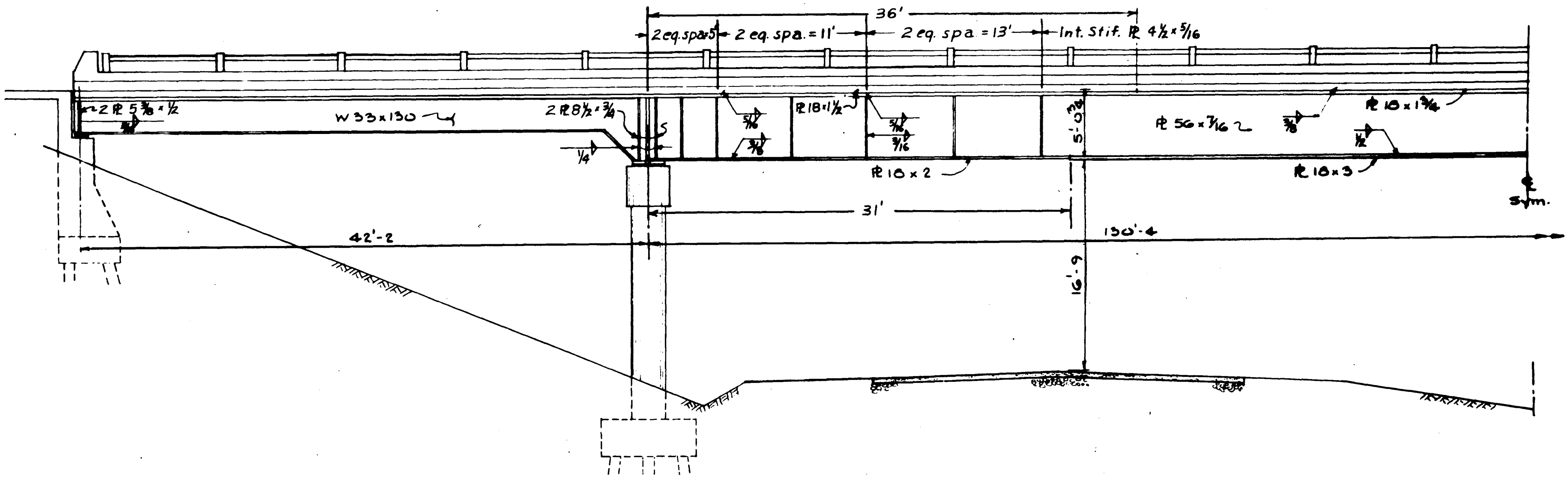


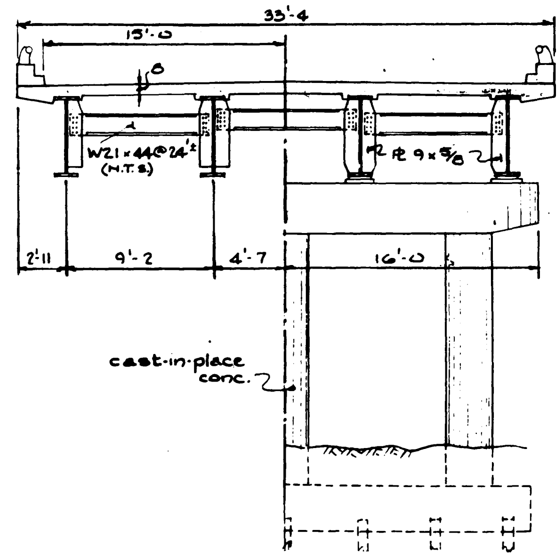
NOTES:

1) See Figure 5 for abutment details
2) See Figure 7 for interior and exterior frame details, detail B, section B-B, and flange transition details taken from bridge plans for Rt. I-64 over Rt. 250 East of Charlottesville, Virginia
3) Provide 0.28 inch (0.71 cm) camber in end spans of frames
4) Provide 2.19 inch (5.56 cm) camber in end spans of frames

Figure 1. Final design for Plan A - RIGID FRAME.

Basic conversions 1" = 2.54 cm; 1' = 0.305 m

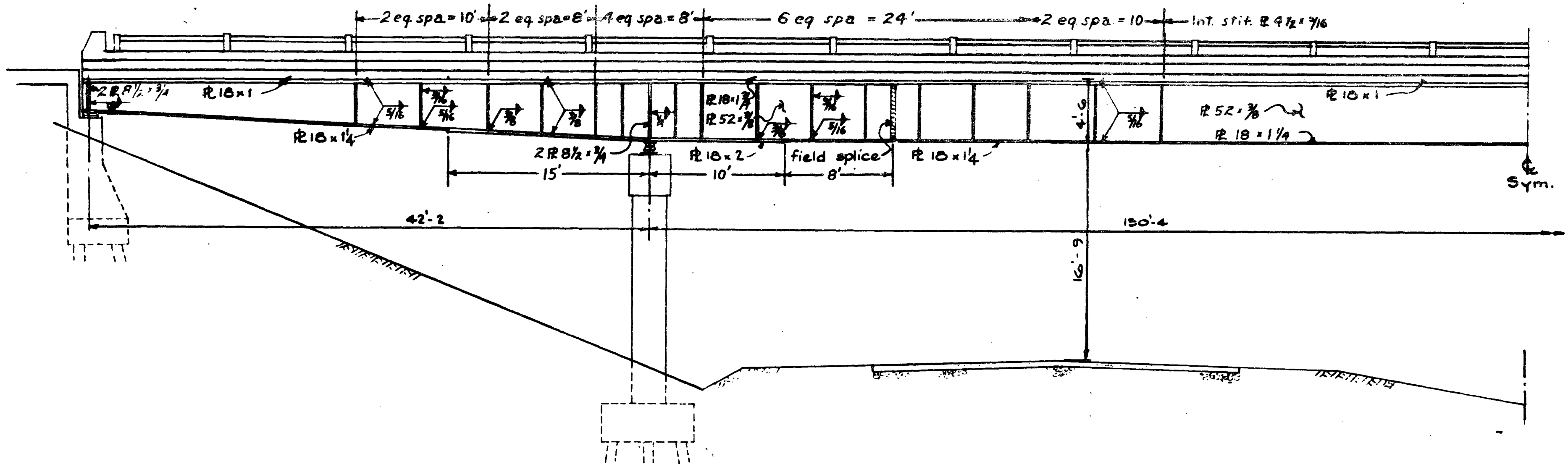


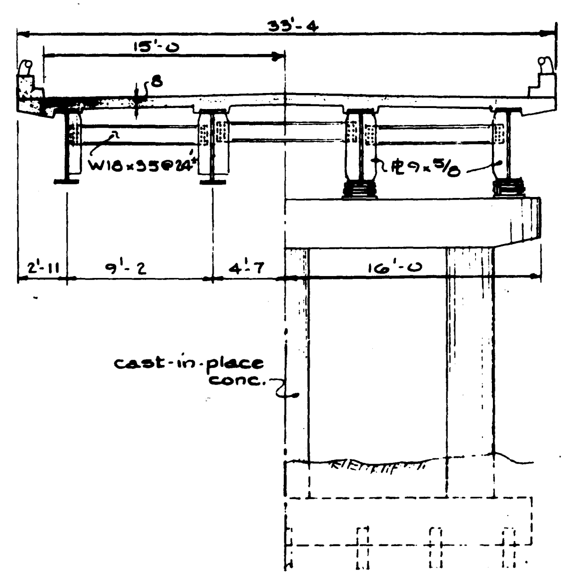


NOTES:

1) See Figures 5 and 6 for substructure details
2) See Figure 7 for bearing details
3) provide 0.23 inch (0.58 cm) camber in end span girders
4) provide 4.18 inch (10.6 cm) camber in center span girders

Figure 2. Final design for Plan B - SIMPLE SPAN.
Basic conversions 1" = 2.54 cm; 1' = 0.305 m





NOTES:

1) See Figures 5 and 6 for substructure details
2) See Figure 7 for bearing and field splice details
3) Provide 0.35 inch (0.89 cm) camber in end spans of girders
4) Provide 2.73 inch (6.93 cm) camber in center span of girders

Figure 3. Final design for Plan C - CONTINUOUS SPAN.
Basic conversions 1" = 2.54 cm; 1' = 0.305 m

Plan D — Space Frame

Since Plans A, B, and C provide center span steel superstructure depths ranging from 45" (114 cm) to 60.75" (154 cm), a 42" (107 cm) center to center vertical spacing of the upper and lower longitudinal members of the space frame was considered appropriate. A 6' (1.83 m) transverse spacing was selected to accommodate the 30' (9.14 m) roadway width and to establish as much as possible an equilateral triangular relationship between the chord weld points. Similarly a 7' 8" (2.34 m) longitudinal spacing was established to accommodate the bridge span lengths and to maintain approximate three-dimensional equilateral compatibility.

After the basic geometric configuration for the space frame was established, available tubular steel sections were studied to determine the most appropriate shape, size, and steel strength for the members. Since a hollow cylinder provides the greatest strength-to-weight relationship for torsion and biaxial bending, pipe sections were selected for chord positions. However, rectangular tubes were chosen for the longitudinal and transverse positions for the following reasons.

(1) The cutting and fitting of the pipe chords could be more easily accomplished.

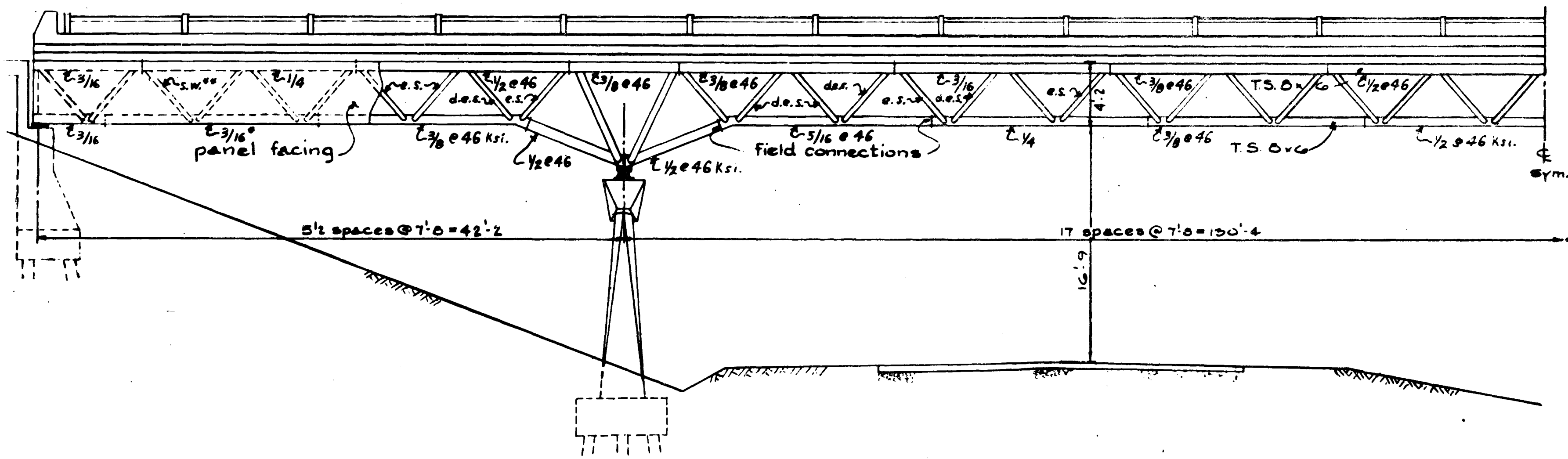
(2) The longer sides of the rectangular tube provide the necessary weld area.

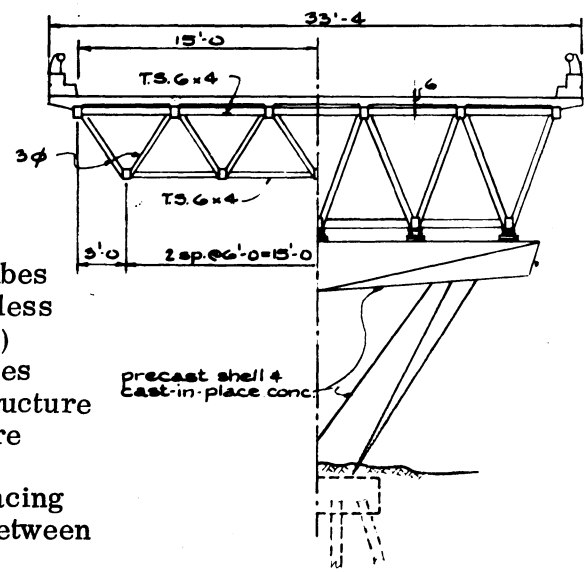
(3) Chord forces are concentrated near the center of the rectangular cross section as assumed in design.

(4) Bending stresses governed by the maximum radius of gyration may be developed in the laterally braced upper longitudinal members.

(5) The modulus of rigidity of a rectangular tube is greater than that of a square tube with a comparable governing radius of gyration.

By providing A500 (46 ksi (317 MPa)) and A501 (36 ksi (248 MPa)) 8" x 6" (20 cm x 15 cm) rectangular tubes and A501 (36 ksi (248 MPa)) 3" (7.67 cm) nominal diameter pipe sections of variable wall thicknesses, fabrication was simplified, over design was held to a minimum and allowable stresses were not exceeded. As shown in Figure 4, the wall thickness and steel strength of the members vary along the span according to design criteria.





NOTES:

1) See Figures 5 and 6 for substructure details
2) See Figure 7 for bearing details
3) * nominal wall thickness (inches) of 8" x 6" (20 cm x 15 cm) tubes
4) ** 3 inch (7.6 cm) pipe sections are standard weight (s.w.) unless specified as extra strong (e.s.) or double extra strong (d.e.s.)
5) Use 1/4 inch (0.64 cm) fillet welds for hubs and for d.e.s. pipes and use 3/16 inch (0.48 cm) fillet welds elsewhere in superstructure
6) All steel is A501 (36 ksi (248 MPa) yield strength) except where noted as different
7) To compensate for dead load deflection, provide camber by placing a 1/16" x 11" x 1' - 1" (0.16 cm x 27.9 cm x 33.0 cm) plate between bottom hubs of end span and top hubs of center span sections.

Figure 4. Final design for Plan D - SPACE FRAME.

Basic conversions 1" = 2.54 cm; 1' = 0.305 m; 1 ksi = 6.895 MPa

A Fortran computer program (Figure B-1, Appendix B) incorporating the classical pin joint stiffness method was developed to design the space frame. Input data included the three-dimensional coordinates and external forces for each joint. an indication of which joints were fixed, and the A501 (36 ksi (248 MPa)) and A500 (46 ksi (317 MPa)) available steel sections of various wall thicknesses from which to select. Because of the CDC 6400 (Control Data Corporation) capacity and the symmetry of the space frame only one-quarter of the total structure was read into the computer. Several iterations were required for each of the various loading conditions. An iteration involved the determination of the force in each member and the selection, based on allowable stresses, of the smallest available sections to accommodate the corresponding forces. The design was complete when two successive iterations produced the same results.

The equivalent concentrated loads used for the various loading conditions are given in Appendix B. The parapet load was assumed to be concentrated on the outside joints of each transverse section. The slab and steel member loads were assumed to have simple support. To produce maximum stress, the standard AASHTO HS20-44 10' (3.05 m) lane load was applied to two locations in a 15' (4.57 m) design traffic lane as shown in Figure B-2 of Appendix B.

The largest sections produced by each of the various loading conditions were incorporated into the final design shown in Figure 4. The four loading conditions (Appendix B) used to determine the final design were the maximum negative moment loads for Case #1 and Case #2, the maximum positive moment load for Case #2, and the maximum shear load for Case #2.

The loading condition described by Case #1 was used only for the maximum negative moment since it was found that Case #1 and Case #2 loadings produced similar structural behaviors. For example, most of the member forces varied by only 0 to 5% as the loading was moved from the outside to the inside of the 15' (4.57 m) design traffic lane. Also, only four of the 265 members changed size and in no case did the change produce a member size which was greater than that of the largest member in the same transverse section. Furthermore, for the various loading conditions, differential deflection of the pin joints along a transverse section did not exceed 1/50" (0.51 mm) and in many instances 1/100" (0.25 mm). Although there was some variation in the member sizes along a transverse section, the general behavior was evidence of a satisfactory distribution of the load throughout a rigid transverse section.

The Fortran design program indicated that the maximum live load deflection of Plan D was 3.3 " (8.38 cm), which exceeds the 1.95" (4.95 cm) allowed for a 130.33' (39.72 m) span. However, the stiffness provided by the composite deck and the torsional resistance provided by the chord members were not considered in the deflection calculation. Further study is needed to determine the exact live load deflection of Plan D. Nevertheless, it is felt that any increases in the depth of the space frame necessary to reduce deflection would not significantly alter the cost estimates.

Substructures

Substructures are rarely prefabricated because design and construction are usually controlled by the geological conditions at the site. Cast-in-place concrete is used for most bridge substructures. In view of these facts and since the emphasis of this project was on steel bridge construction, only limited consideration was given to substructure design. The legs of Plan A represent the extent of steel substructure considered in this project. Nevertheless, the design of a conventional substructure, compatible with the superstructures shown in this report, was necessary for the satisfactory completion of the project. The design for conventional cast-in-place piers and abutments is shown in Figure 5.

To accompany the industrialized superstructure presented in this project, it was considered appropriate to provide a precast alternative substructure design. Therefore, along with some additional space frame details, an innovative precast shell assembly with posttensioned cap beam is shown in Figure 6. The basic design for either of the alternative substructures can be used with any of the superstructures with the exception of Plan A.

Additional Details

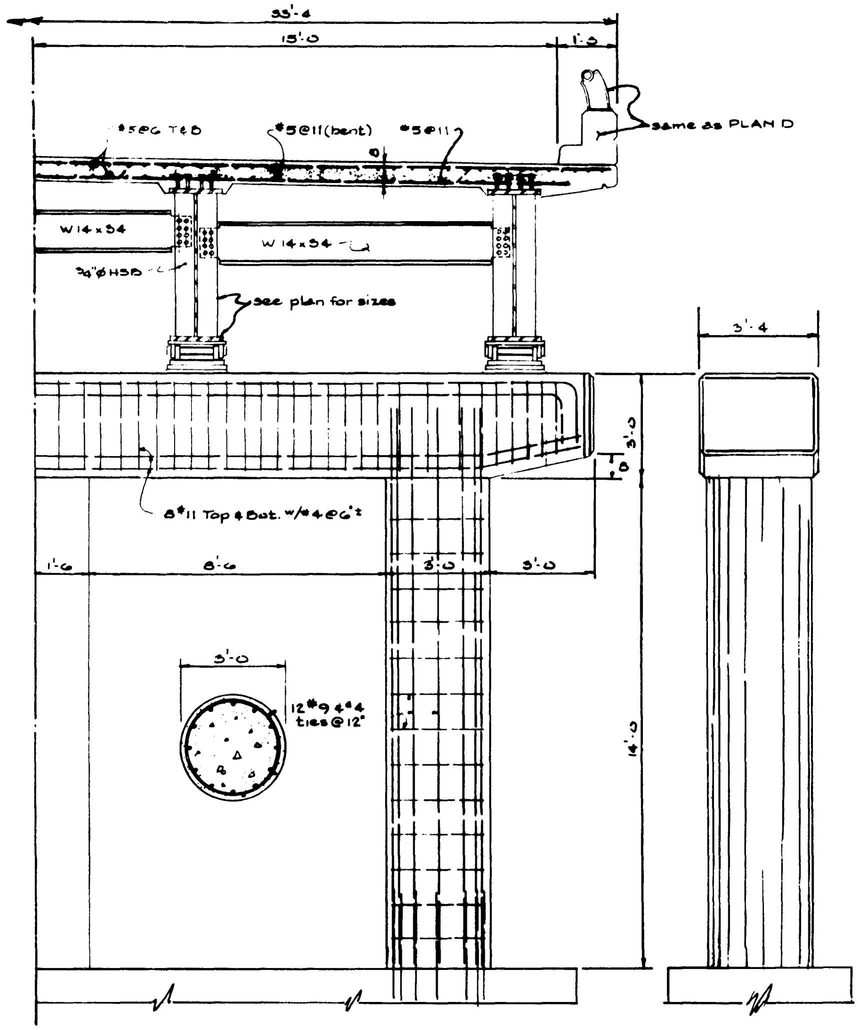
In addition to the typical details shown in Figures 1-6, each of the fabricators and contractors was provided with the itemized quantitites shown in Table 1. Also, at the request of one of the steel fabricators providing cost estimates, the additional design details shown in Figure 7 were developed.

TABLE 1
Summary of Quantities for Plans A, B, C, and D

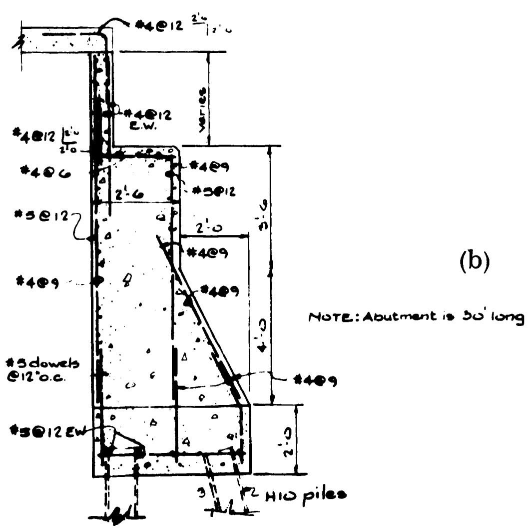
ITEM \ PLAN			A	B	C	D
A3 Concrete		CY	100	140	139	130 (a)
A4 Concrete		CY	228	228	228	184
Excavation		CY	700	700	700	700
Reinforcement		LB	63,200	71,000	71,000	36,200
Structural Steel	(b)	LB	218,100	239,300	208,700	137,600
Railing		LF	426	426	426	426
Steel Piling		LF	2,480	2,240	2,240	2,080
Concrete Slope		SF	400	400	400	400
Precast Piers	(c)		0	0	0	10

a. For precast pier alternative assume 78 CY (60 m^3) A3 and 40 CY (31 m^3) A5 concrete.
b. Includes 54,700 lbs. (24,800 kg) 46 ksi (317 MPa) steel in PLAN D and 5,600 lbs. (2,540 kg) of steel for bearing assemblies and expansion joints.
c. Assume 20 reuses of forms.

Basic conversions: CY = 0.765 m^3 LF = 0.305 m
LB = 0.4536 kg SF = 0.093 m^2



(a)



(b)

Figure 5. Conventional cast-in-place substructure design.
(a) Typical cross section Plans B and C
(b) Typical abutment Plans A, B, C, and D

Basic conversions 1" = 2.54 cm; 1' = 0.305 m

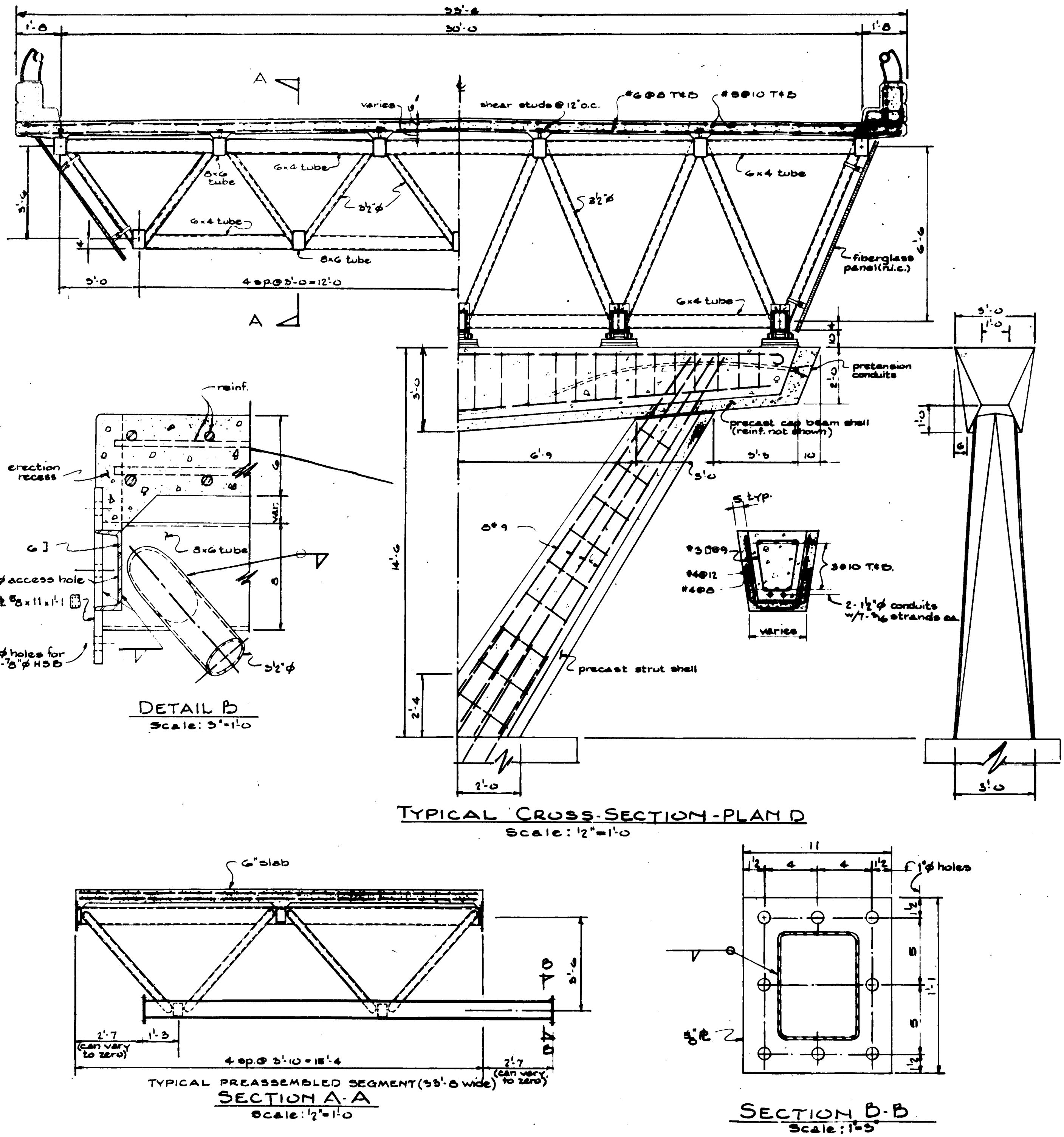


Figure 6. Precast shell assembly with posttensioned cap beam shown with space frame.

Basic conversions 1" = 2.54 cm; 1' = 0.305 m

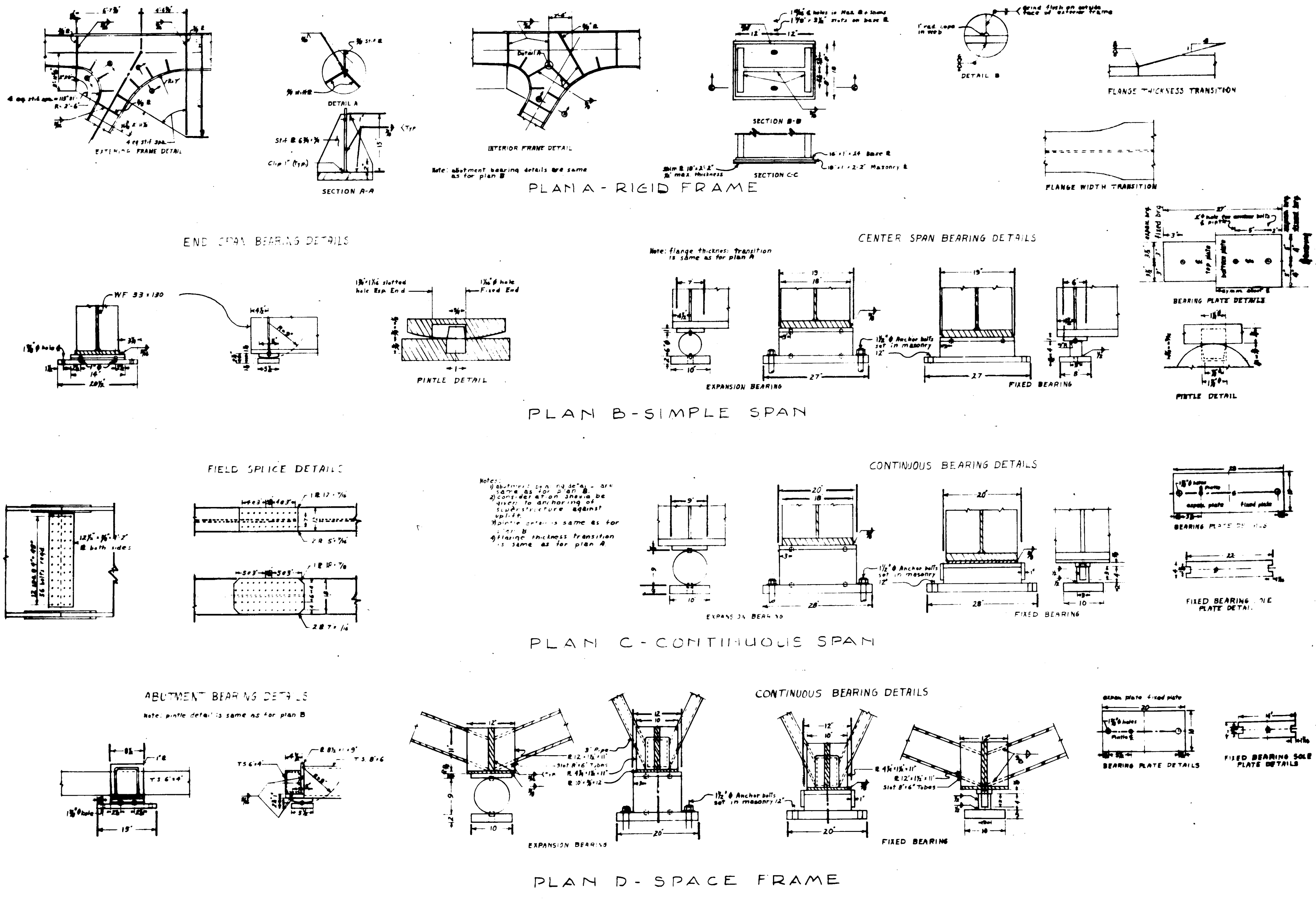


Figure 7. Additional details.

Basic conversion 1'' = 2.54 cm

FABRICATION

The four structures investigated represent a range of fabrication and material requirements. Plan B combines simple plate girders and standard wide flange sections to provide minimal fabrication intricacy but maximum material quantity. The tapered girders and numerous stiffeners required for Plan C increase fabrication time but reduce material quantity. The curved haunch section required for Plan A increases fabrication time but provides aesthetic appeal and material economy. Plan D requires the most fabrication time per pound but the least material quantity, and represents an effort to incorporate the fabrication simplicity of Plan B and the material economy of Plans A and C. It should be noted that the fabrication and design time for Plan A would increase considerably if the rigid frame is designed for a grade. This increase in fabrication and design time would not apply to Plans B, C, and D, in which the symmetry of the superstructure is not affected by grade.

The fabricators who provided cost estimates for the four structures differed in their opinions as to relative material costs and levels of fabrication intricacy. Nevertheless there was basic agreement on certain points. For example, standard wide flange sections and simple lines tend to minimize fabrication time, whereas stiffners and curved or tapered girders tend to increase fabrication time. Also, the material cost per pound is considerably more for tube and pipe sections than for plate material. This basic agreement was exemplified in the final cost estimates, which indicated that fabrication costs per pound increase in the following order from lowest to highest — Plan B, C, A, and D.

Although the basic trends noted above were apparent from the estimates, the relative differences in cost between the four structures depended on the fabricator's shop layout, labor supply, and familiarity with the type of fabrication. The diversities in shop layout and the differences in weight among the four designs resulted in a considerable range in the relative estimates of the four fabricators. For example, fabricator #2 (see Table 2), who was set up for low labor, plate type fabrication, estimated the space frame fabrication cost at 1.9 times the fabrication figure for the simple span. Fabricator #3, who has done considerable space frame construction for the building industry, estimated that the space frame could be fabricated for the same cost as the simple span.

TABLE 2
Fabrication Cost Ratio of the Space Frame to the Average of Plans A, B, and C

Ratio \ Fabricator	1	2	3	4
Labor $\frac{D}{(A+B+C)/3}$	1.47	2.00	1.03	1.30
Material $\frac{}{(A+B+C)/3}$	1.10	1.63	1.01	0.90

These four fabricators were instructed to make their estimates on the basis that ten similar bridges would be built. Based on these estimates, the ratios of the labor cost and material cost of the space frame to the average respective costs of Plans A, B, and C are shown in Table 2. In general, it can be concluded that a ratio of 1.25 for labor and 1 for material could be readily obtained and that a ratio of approximately 1 for both labor and material might be possible.

At the onset of this project it was felt that fabrication costs might eliminate the space frame as a competitive alternative to bridge construction. As a result considerable effort was made to hold fabrication costs to a minimum in the multimember space frame. For example, the space frame consists of four primary types of members: 8" x 6" (20 cm x 15 cm) rectangular tubing, 3" (7.6 cm) nominal diameter pipe, 6" x 4" (15 cm x 10 cm) rectangular tubing, and 6" (15 cm) channels. Also, with the exception of the members over the support, similar types of members are the same length. Although the chord members require a skew cut, most of the skews are identical and most of the other members can be cut at 90°. It is envisioned that a cutting jig and an adjustable holding frame could provide for systematic fabrication. The estimates verify that the space frame can be economically fabricated.

Precast Deck

Plan D is designed to be erected in transverse sections with the deck already cast to the steel frame. Forms must be placed to provide a uniform 6" (15 cm) thick deck with standard slope for drainage and the deck should be cast with the steel frame in an upright position. The same forms could be used for each casting operation.

For handling purposes it is suggested that four lift hook inserts (Figure 8) be installed in each space frame section prior to casting the deck. The inserts should be welded to the outer edge of the upper outside longitudinal rectangular tubes just behind the hubs and the deck should be cast around the inserts. Once a space frame section is in place and the lift hooks have been removed the openings can be filled with concrete as the parapets are cast or prior to attaching precast parapets.

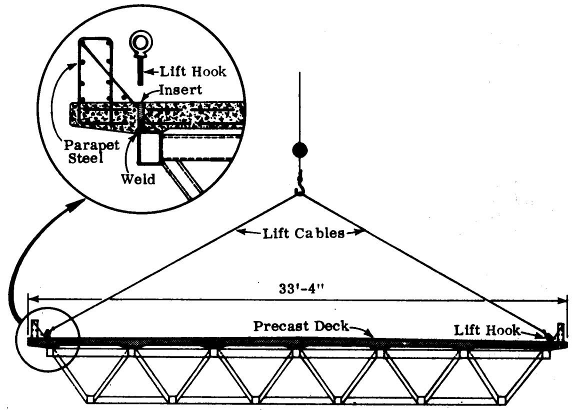


Figure 8. Space frame handling hooks.
Basic conversion 1' = 0.305 m.

TRANSPORTATION

Based on the fabricators' estimates, Plans A, B, and C could be shipped for the same cost. However, a splice not shown on the final design details of Plan B was required to permit hauling of the center span girders.

It was anticipated that the space frame sections as designed for Plan D could be hauled as shown in Figure 9 by providing the appropriate offset distance "X" to accommodate the hauling vehicle.

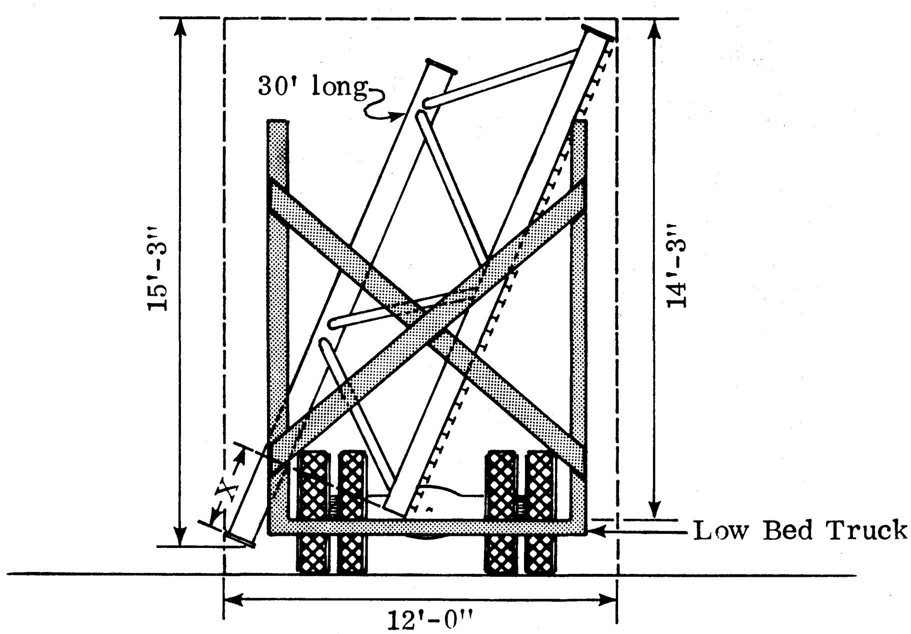


Figure 9. Hauling arrangement for space frame sections. X = offset distance between upper and lower hubs. Basic conversion 1' = 0.305 m.

The fabricators indicated that a special rig would be required to haul the sections as shown in the figure. Assuming that clearance and weight permits could be obtained, the fabricators estimated that the space frame sections without the deck could be hauled for three times the cost of each of the other plans. Although this is a significant difference it is worthy of note that the transportation cost for a 100 mile (161 km) radius from a fabricating plant was only approximately 1.25% of the average fabrication cost of Plans A, B, and C.

The feasibility of helicopter transportation of the space frame sections was investigated.[14] It was found that maximum helicopter lifting capacities ranging from 9,000 - 18,000 lbs. (4080 - 8160 kg) could handle the steel space frame sections. However, with the near capacity loading, fuel consumption would not permit hauls of more than several miles. Furthermore, once the deck was cast onto the section, the helicopter capacity would be exceeded. Finally, strict federal regulations governing air transportation of sling loads combined with the distance and capacity limitations of the helicopter tend to make helicopter transportation unfeasible.

After further consideration, it was felt that the space frame section length should be reduced for the following reasons. First, few bridges provide 16' - 9" (5.11 m) clearances. Secondly, the use of a special hauling rig is undesirable. Third, once the deck is cast onto the steel frame, stability during handling and shipping could present problems. Finally, once the deck is in place, the average weight of a section will reach 47,600 lbs. (21600 kg) (based on 150 pcf (2400 kg/m^3) deck), which could result in overloads on certain highways.

The initial space frame section length had been established to minimize the number of sections, field connections, and transverse deck joints, while providing for satisfactory handling, hauling and maintaining the initial structural geometry. However, to eliminate the disadvantages mentioned above, a 12' - 0" (3.66 m) standard section as shown in Figure 10 should provide a satisfactory compromise between handling and shipping requirements and the desire to minimize the total number of sections. With this configuration the sections could be hauled on a standard truck bed as shown in Figure 11. The total weight of a standard 12' - 0" (3.66 m) section would be slightly less than 40,000 lbs. (18100 kg), which is within the allowable highway load limit. Although wide load permits would be required, the weight and height would not present problems. The need for a special rig would be eliminated and as many as six sections without decks could be hauled in one trip. With the 12' - 0" (3.66 m) standard section configuration, transportation costs for the space frame should not exceed those of Plans A, B, and C. Also, the center support sections of the space frame would be almost standard, which favors fabrication, transportation and erection.

Stability during transportation could be improved by hauling the space frame sections upside down. However, there are numerous disadvantages to hauling the sections in this manner. First, they would have to be inverted prior to erection. The inversion process would be time-consuming and could result in damage to the sections. Secondly, transportation supports of varying thickness would be required to accommodate the slope of the precast deck surface. Finally, the reinforcing bars which protrude from the deck would probably have to be bent to accommodate hauling. The reshaping of bent bars prior to casting the parapets would be a time-consuming and costly task.

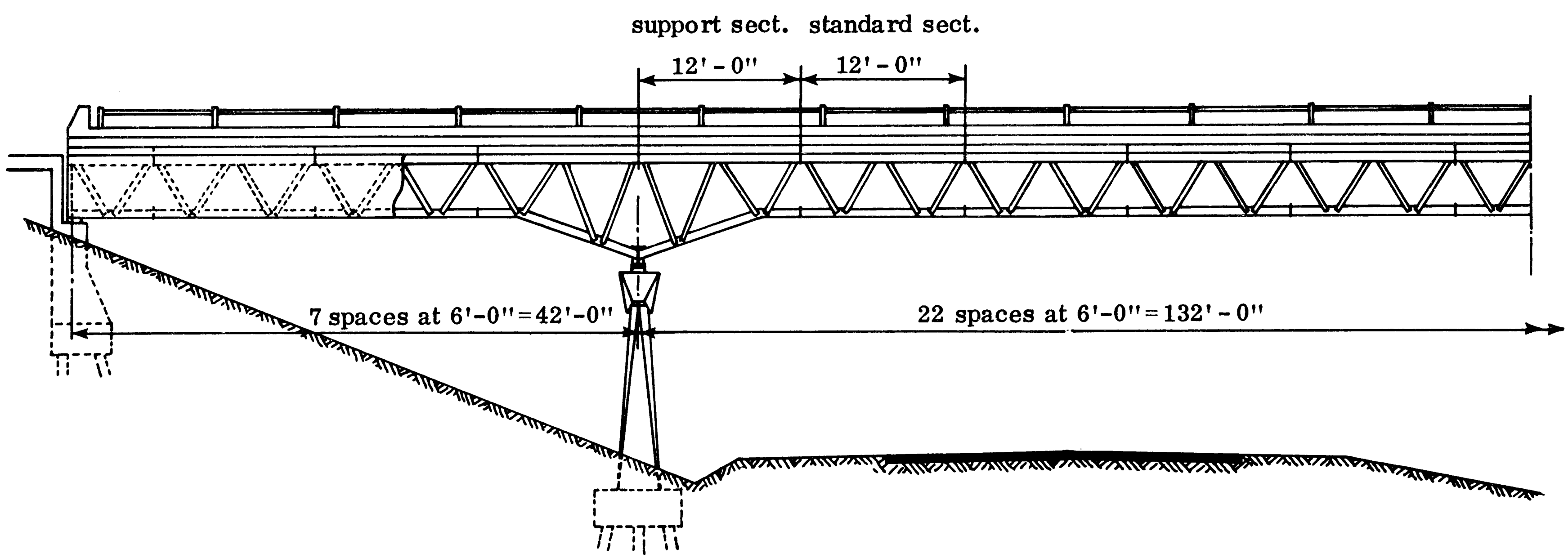


Figure 10. Revised space frame design with standard 12' - 0" (3.66 m) sections

Basic conversion 1' = 0.305 m

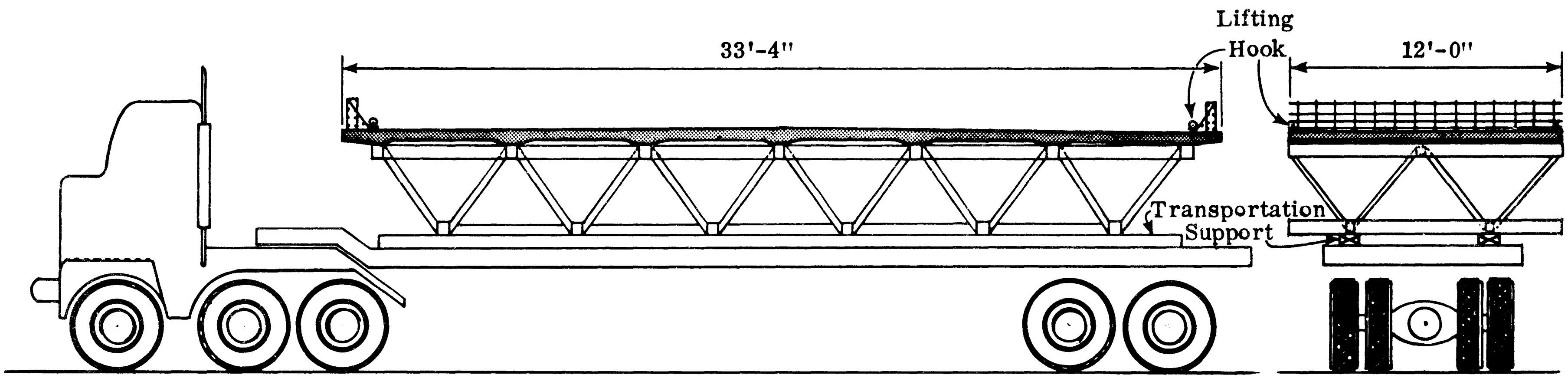


Figure 11. Hauling arrangement for 12' - 0" (3.66 m) sections

Basic conversion 1' = 0.305 m

Precasting the parapets would eliminate the possibility of bending or damaging the protruding reinforcing bars during the handling of the space frame sections. The expensive form work required for cast-in-place parapets would also be eliminated. However, precasting the parapets as an integral part of the precast deck is not recommended because the weight of the 12' - 0" x 33' - 4" (3.66 m x 10.16 m) space frame sections would be increased approximately 6 tons (5400 kg) and also because of the possibility of damage to the parapets during handling. Providing inserts in the precast deck and attaching the precast parapets after erecting the space frame sections are recommended.

CONSTRUCTION

Five contractors in Virginia were contacted to obtain cost estimates and other details related to the construction of each of the four bridge structures shown in this report. However, for various reasons only two responses were received (see Figures 12 and 13). Both of the responding contractors were familiar with the construction shown in Plans A, B, and C, but were forced to rely on their best judgement when assessing Plan D.

The contractors' estimates revealed that the construction cost for any one of the conventional plans would not differ more than 3% from the average cost of the three plans. However, as could be expected, there was some disagreement as to the relative cost of Plan D. Based on the estimates received from the two contractors the ratio of the construction cost of the space frame to the average construction cost of Plans A, B, and C is shown in Table 3.

TABLE 3
Construction Cost Ratio of the Space Frame to the Average of Plans A, B, and C

Contractor Number / Construction Cost Ratio *	1	2
$\frac{C}{(A+B+C)/3}$	0.97	1.09

Contractor No. 1 stated that he felt Plan D would be particularly appropriate for construction in remote areas where the unit cost of cast-in-place concrete would be extremely high. However, when estimating Plan D, neither contractor considered the economic value of reduced on-site construction time. Based on the two estimates the most that can be concluded is that Plan D can be competitively constructed.

* Excluding structural steel fabrication cost.

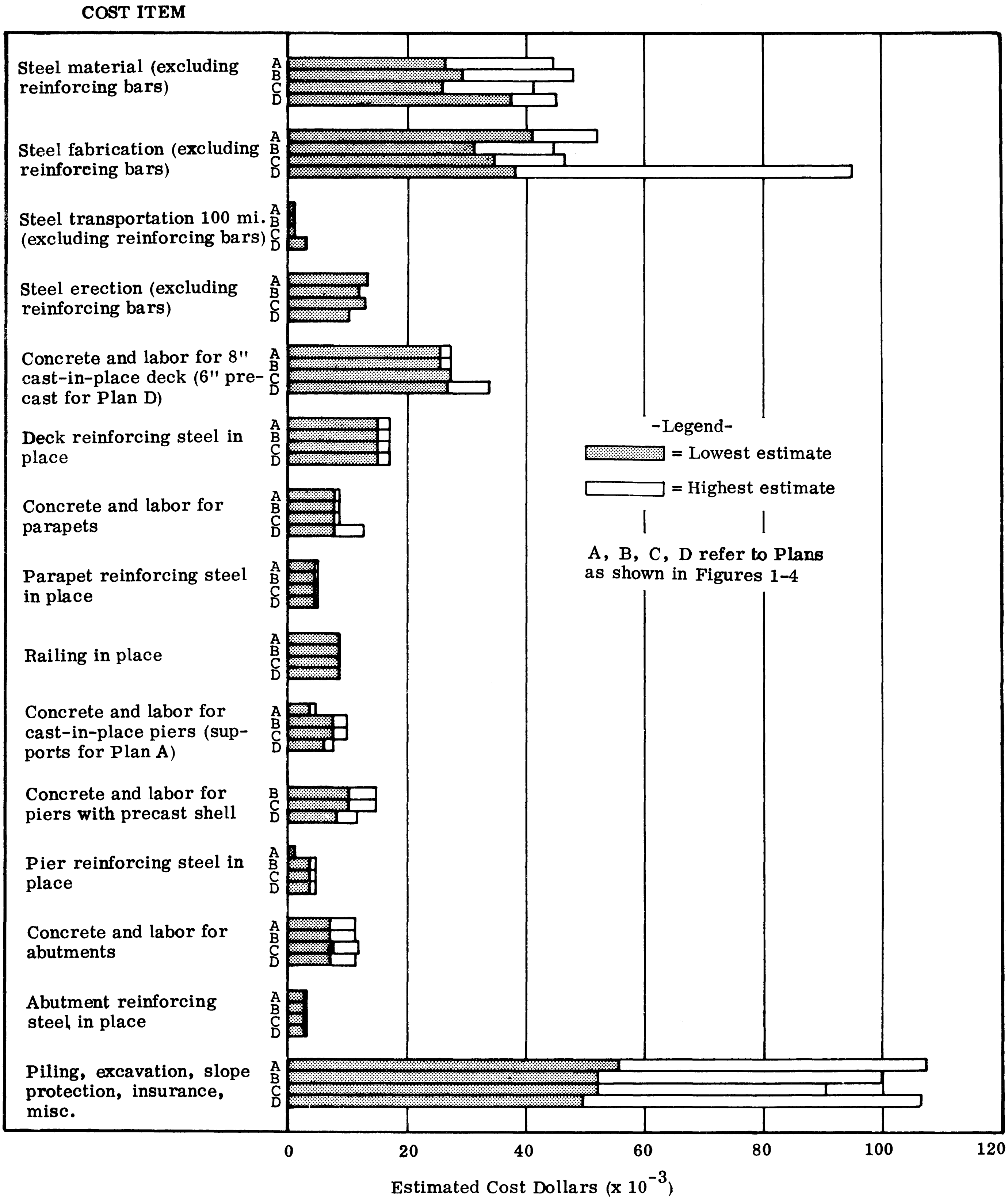


Figure 12. Quantity cost estimates for Plans A, B, C, and D.

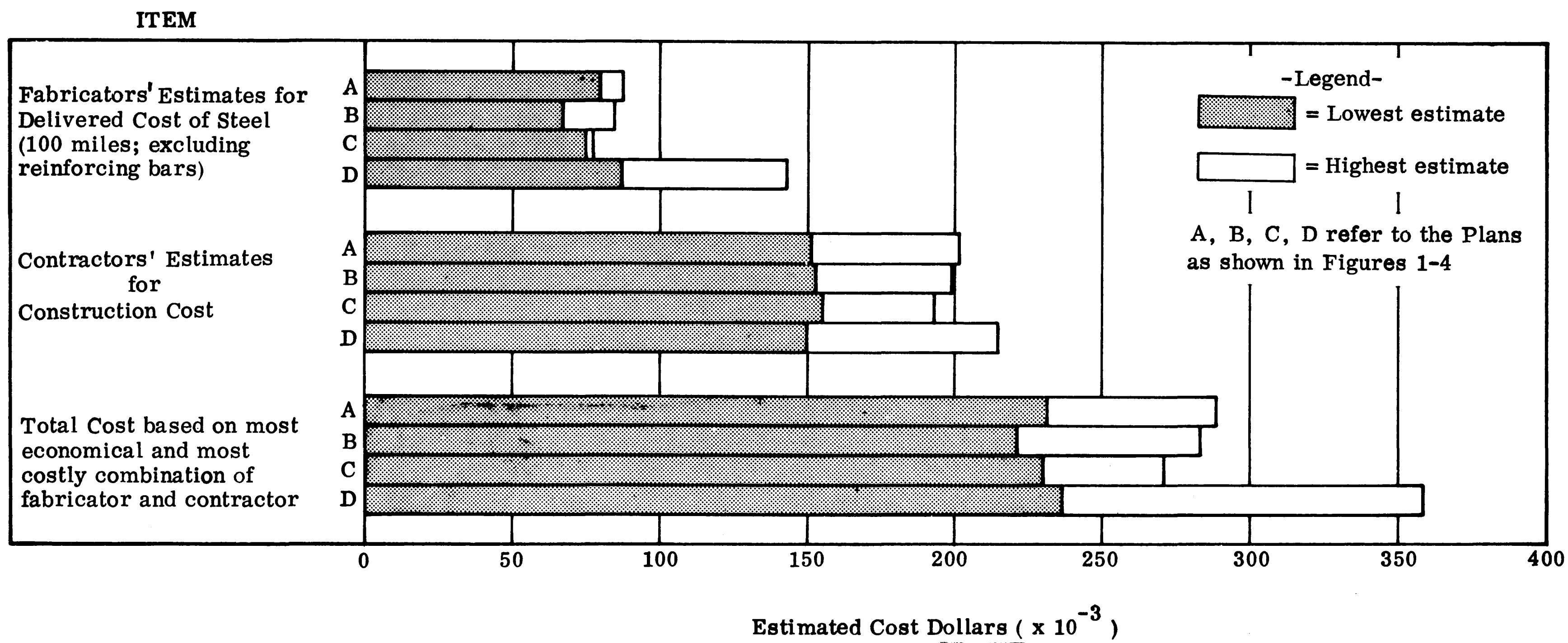


Figure 13. Summary of the cost estimates for Plans A, B, C, and D.

An examination of the estimates from both fabricators and contractors revealed that for Plans A, B, and C, the total bridge costs can be divided approximately equally among the following three areas.

1) fabrication and material costs for superstructure steel (excluding reinforcing bars)

2) construction and material (other than Item 1) costs for superstructure

3) construction and material costs for substructure and other miscellaneous items such as placing slope protection.

By considering the precasting of the space frame deck as part of Item 2, the 1:1:1 relationship shown above was also evident in some but not all of the estimates for Plan D. However, this relationship should not apply to the space frame if the economic value of reduced on-site construction time is considered.

To avoid confusion precast parapets were not included in the plans sent to the contractors. Nevertheless, it is felt that precasting the parapets like precasting the deck should reduce on-site construction time and cost. Also since the estimates revealed that cast-in-place parapet costs for Plan D are greater than for the conventional plans because conventional form supports cannot be used with the space frame superstructure, precast parapets are even more desirable.

The contractors considered precast piers to be slightly more expensive than cast-in-place piers but the difference in cost was less than 2% of the total construction cost. The higher cost may have been a result of the contractors' assumption that they would precast their own piers rather than contract with a prestressed concrete producer for piers which would be cast in a standard reusable form. Cost estimates from prestressed concrete producers were not obtained.

It was anticipated that the elimination of form work and cast-in-place concrete and the corresponding reduction in site time should result in favorable construction cost estimates for Plan D. Obviously, the time and material saving qualities of Plan D did no more than offset the increase in estimates caused by apprehension about an unfamiliar concept. Nevertheless, the author envisions that with experience, Plan D could be constructed much faster and more economically than Plans A, B, and C.

The following construction procedure for the space frame is recommended (Figure 14).

1) Complete the substructure

2) Erect the necessary false work around a center support

3) Using a crane, place and connect two support sections in their appropriate locations. Camber plates should be placed between the appropriate hubs before inserting the bolts, and a neoprene pad should be placed in the transverse deck joint prior to tightening the bolts which connect the sections.

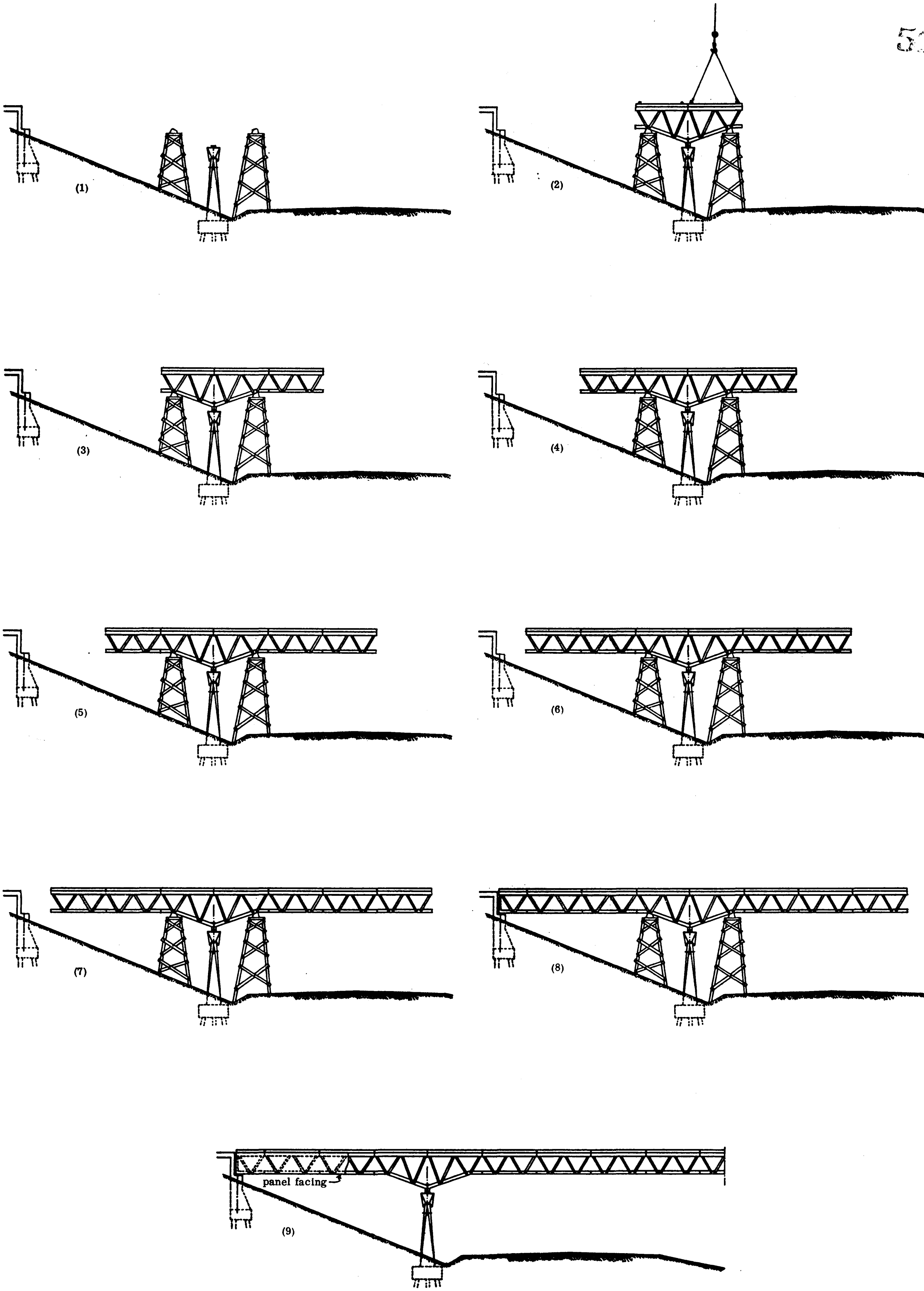


Figure 14. Erection procedure for space frame.

4) In a similar manner erect and bolt into place adjacent sections in an order which will tend to maintain stability about the center support.

5) Continue until an abutment section is in place.

6) Move the false work to the other center support and repeat steps 2-5.

7) Erect the mispan sections and bolt into place.

8) Attach the precast parapets and the panel facing.

CONCLUSIONS

Minimal weight and quick assembly are essential requirements for industrialized construction, and are basic features offered by the space frame concept. Repetitive nature is an economically desirable characteristic for industrialized construction and a basic feature required for the economical fabrication of a multimember structure such as the space frame. The high strength steel and high quality precast deck offered by the space frame concept are benefits typically associated with industrialized construction. In an economy controlled by scarce resources, the material and time savings offered by the space frame cannot be overlooked. Current estimates indicate the space frame can be competitively constructed. The economic value of reduced on-site construction time was not included in these estimates. With implementation, fabricators and contractors should become more familiar with the space frame concept and with time, by its very nature, the preassembled space frame should become an established form of bridge construction. Industrialized construction with steel and the preassembled space frame should become analogous terms.

RECOMMENDATIONS

The prototype structure shown in this report was designed to determine the economic feasibility of a preassembled space frame. Reduced on-site construction time is an additional benefit not reflected in the economic analysis. Nevertheless, the final quantitative results indicate that the concept can be recommended as a potentially competitive type of bridge construction.

Based on research to date, it is recommended that the space frame should be fabricated in standard 12' (3.66 m) sections and with the aid of reusable forms the deck should be precast onto the units while in an upright position. It is further recommended that the preassembled units should be hauled and erected in upright position, and precast parapets should be attached to complete the structure.

It should be noted however, that certain specific areas are in need of further study prior to moving into the fabrication of a prototype structure. These areas include secondary stresses, structural vibration and fatigue. Also, it is suggested that further consideration be given to the development of the most economical repetitive superstructure geometry. Although it is worthy to note that by varying the wall thickness and steel strength of the space frame members a wide range of span lengths can be accommodated with a single geometry, it may well be that a superstructure geometry other than that reflected in this report will be more appropriate from a systems point of view. Following the satisfactory completion of these suggested studies, it is felt that the field implementation of a prototype structure is a necessary prerequisite to making a satisfactory final recommendation on the preassembled space frame concept. Future field implementations will depend on the success of a prototype installation and the economic benefits that can be derived from in-house fabrication and reduced on-site construction time.

ACKNOWLEDGEMENTS

The author is indeed grateful to the steel fabricators and the bridge contractors in Virginia who provided the free cost estimates and other helpful information necessary for the satisfactory completion of this report.

Also, Dr. David Morris should be commended for his timely guidance and for providing many of the basic concepts and ideas reflected by this report. The general assistance provided by both Dr. Morris and Harry Brown is greatly appreciated.

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APPENDIX A

Moment and Shear Curves used for the Design of Plans A, B, and C

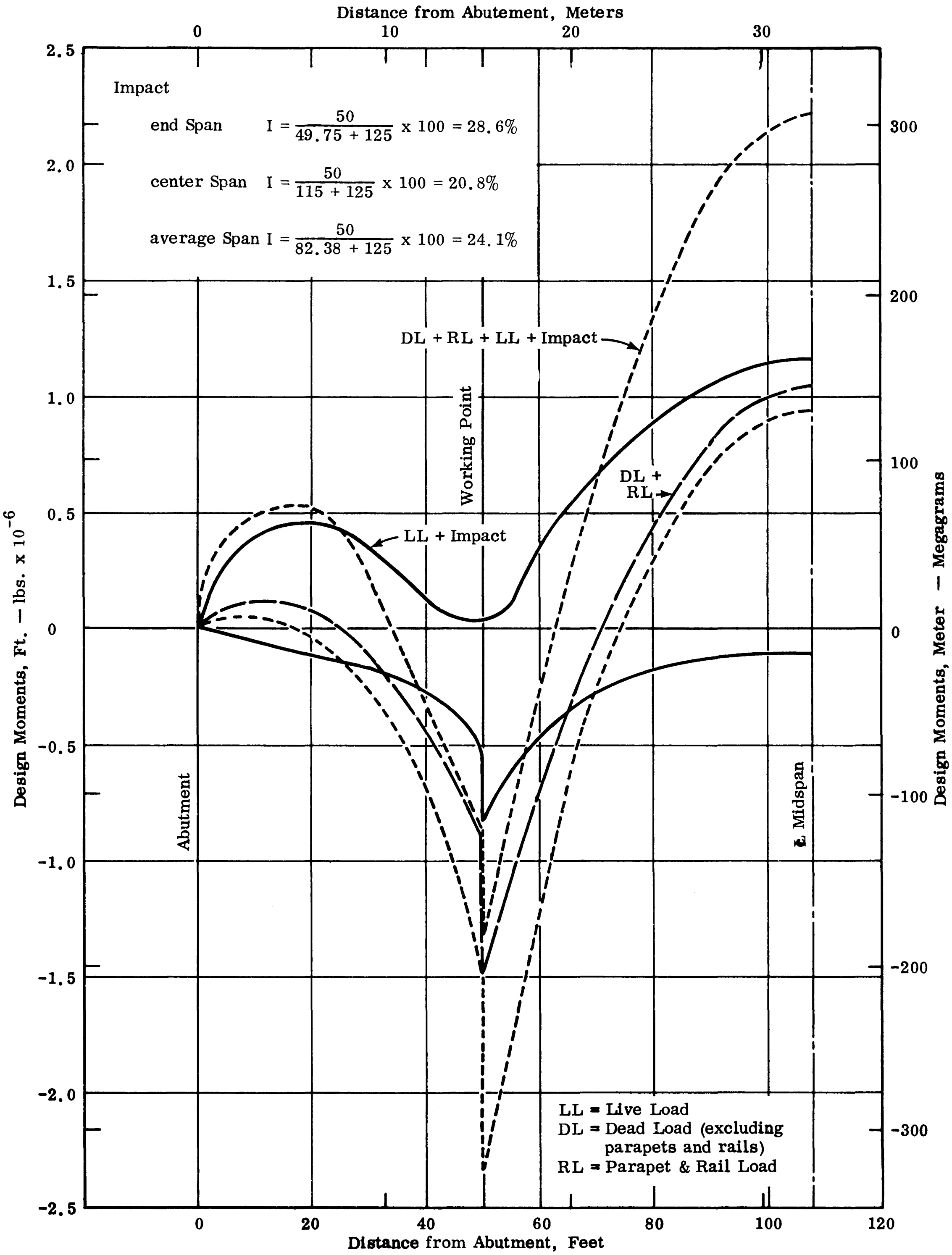

Figure A-1. Moment curves for rigid frame.

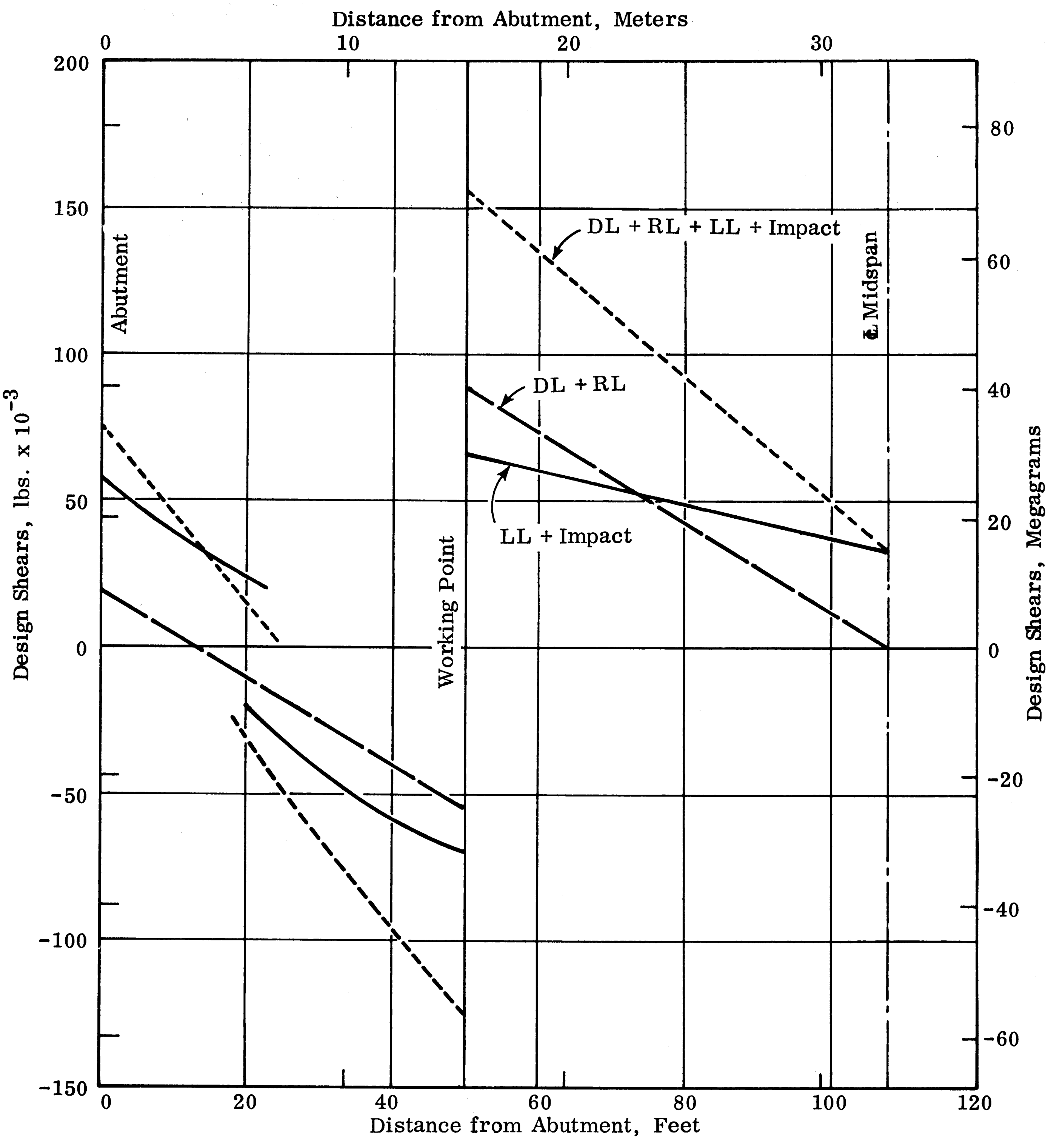

Figure A-2. Shear curves for rigid frame.

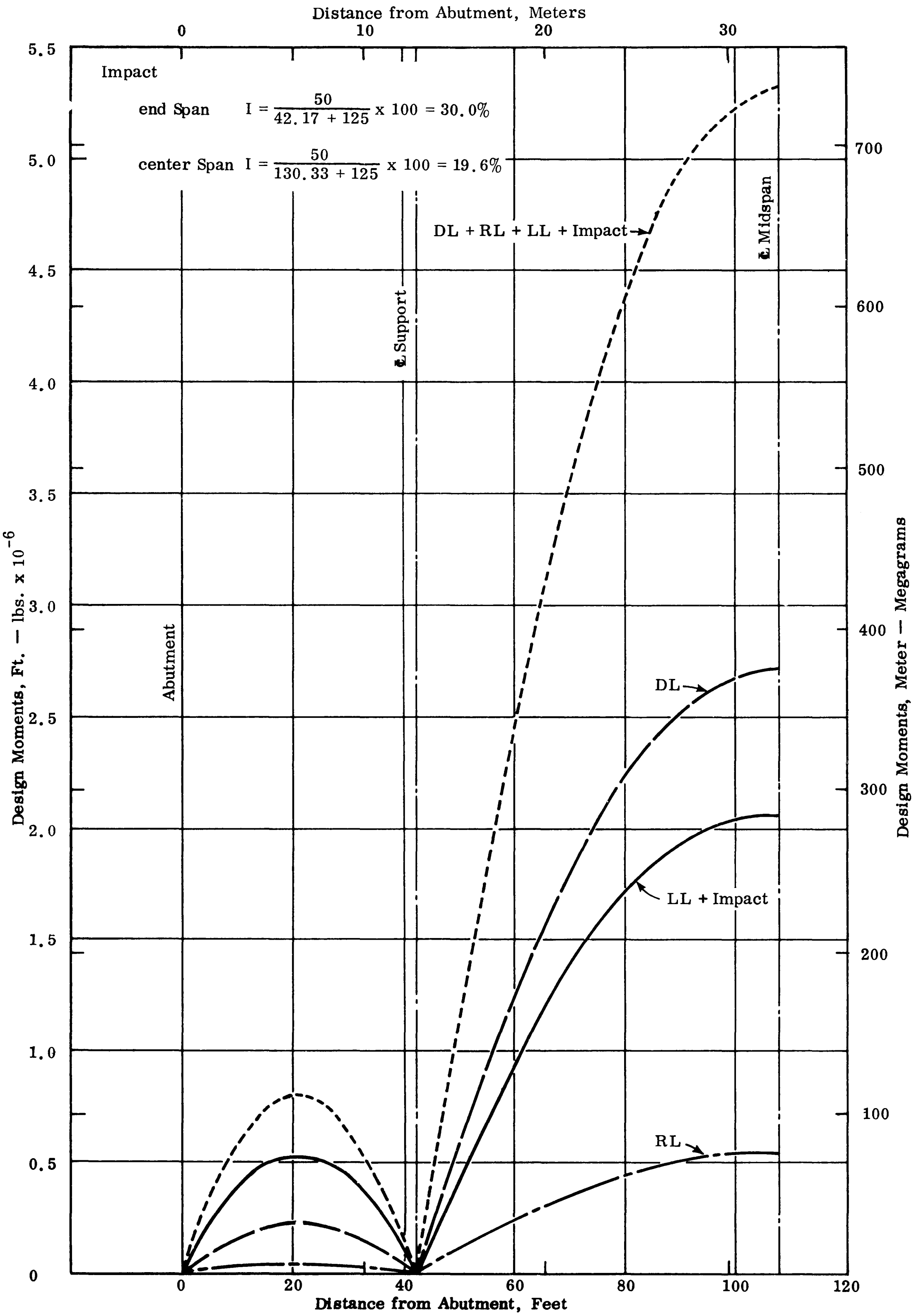

Figure A-3. Moment curves for simple span.

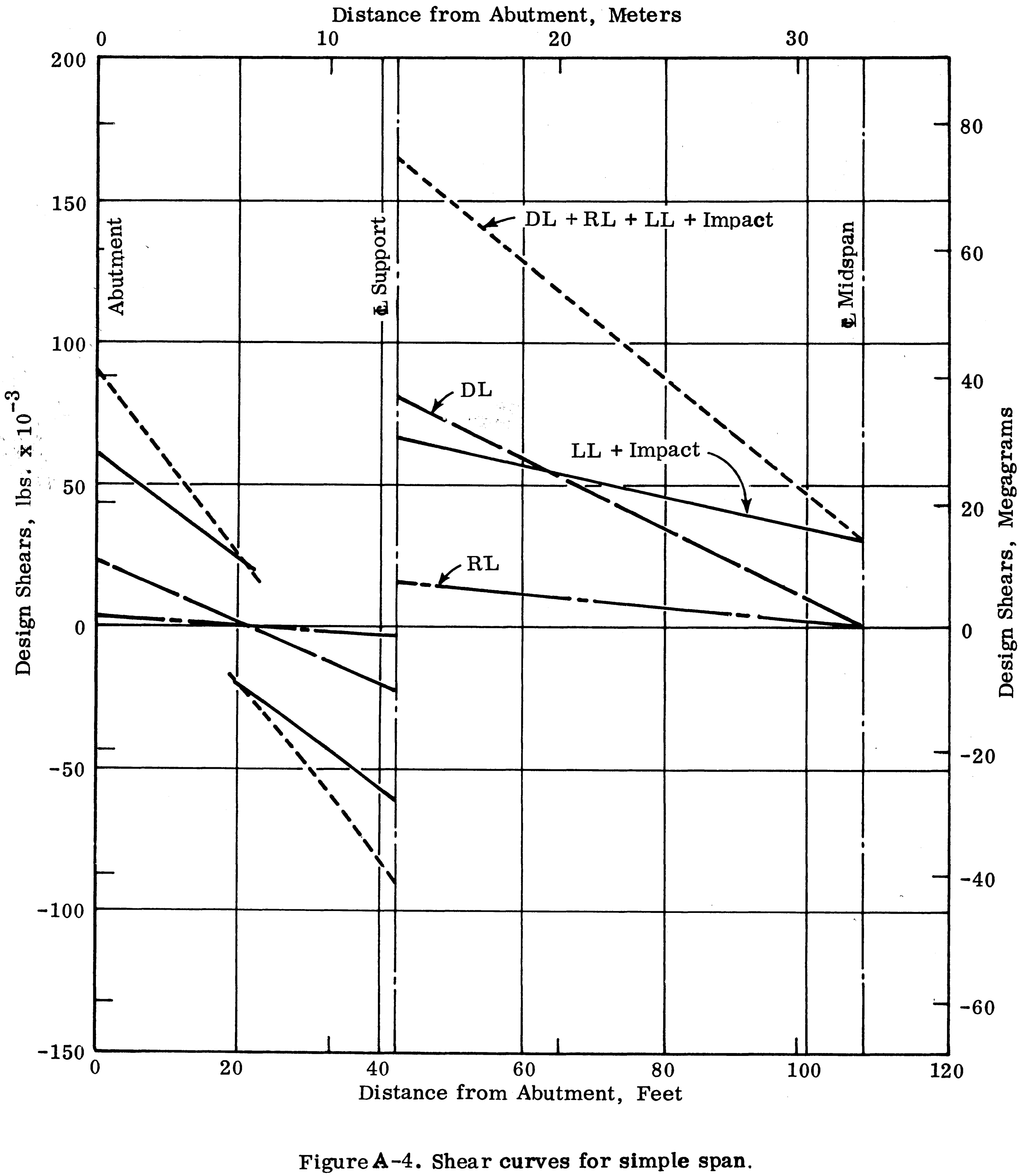

Figure A-4. Shear curves for simple span.

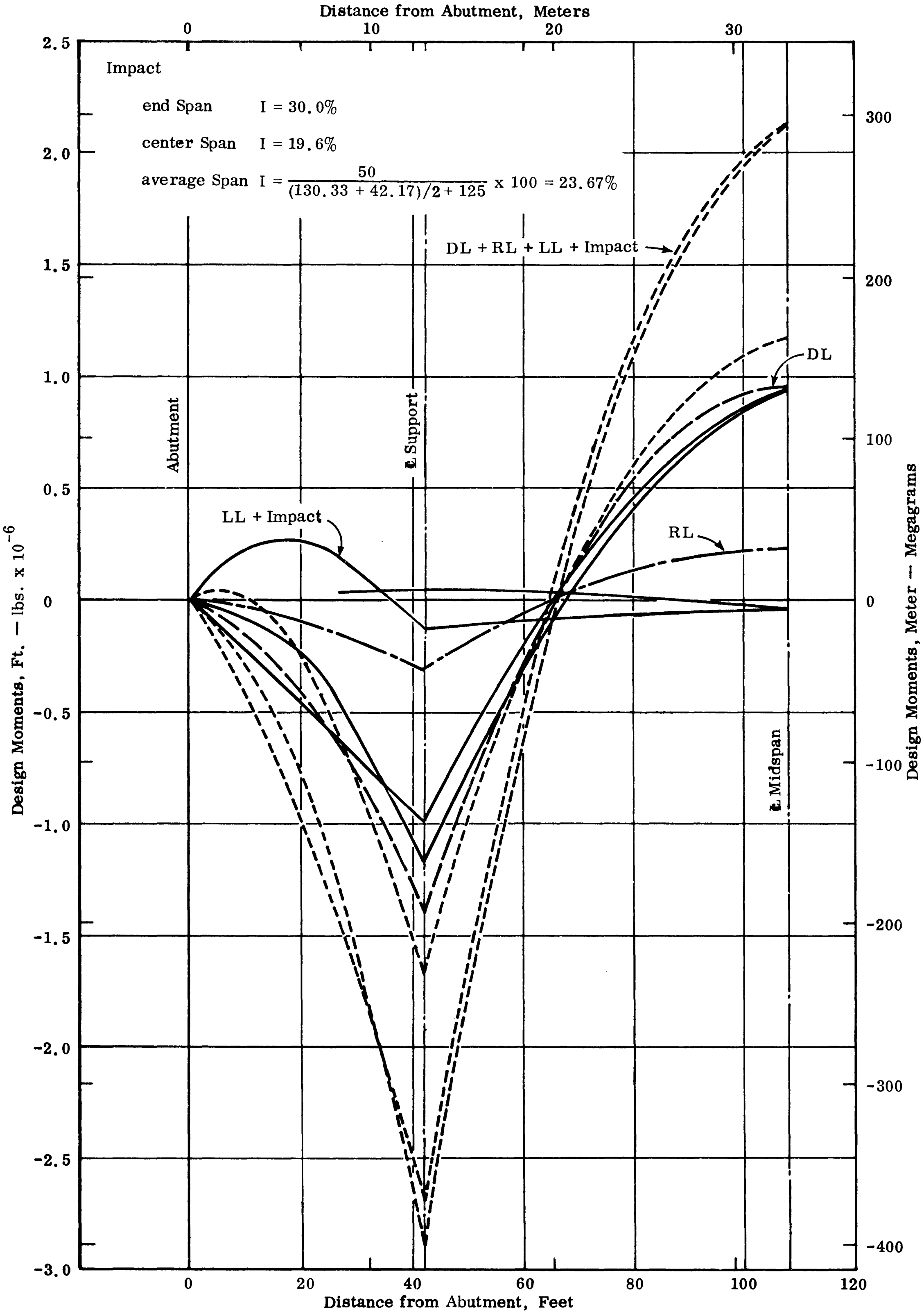

Figure A-5. Moment curves for continuous span.

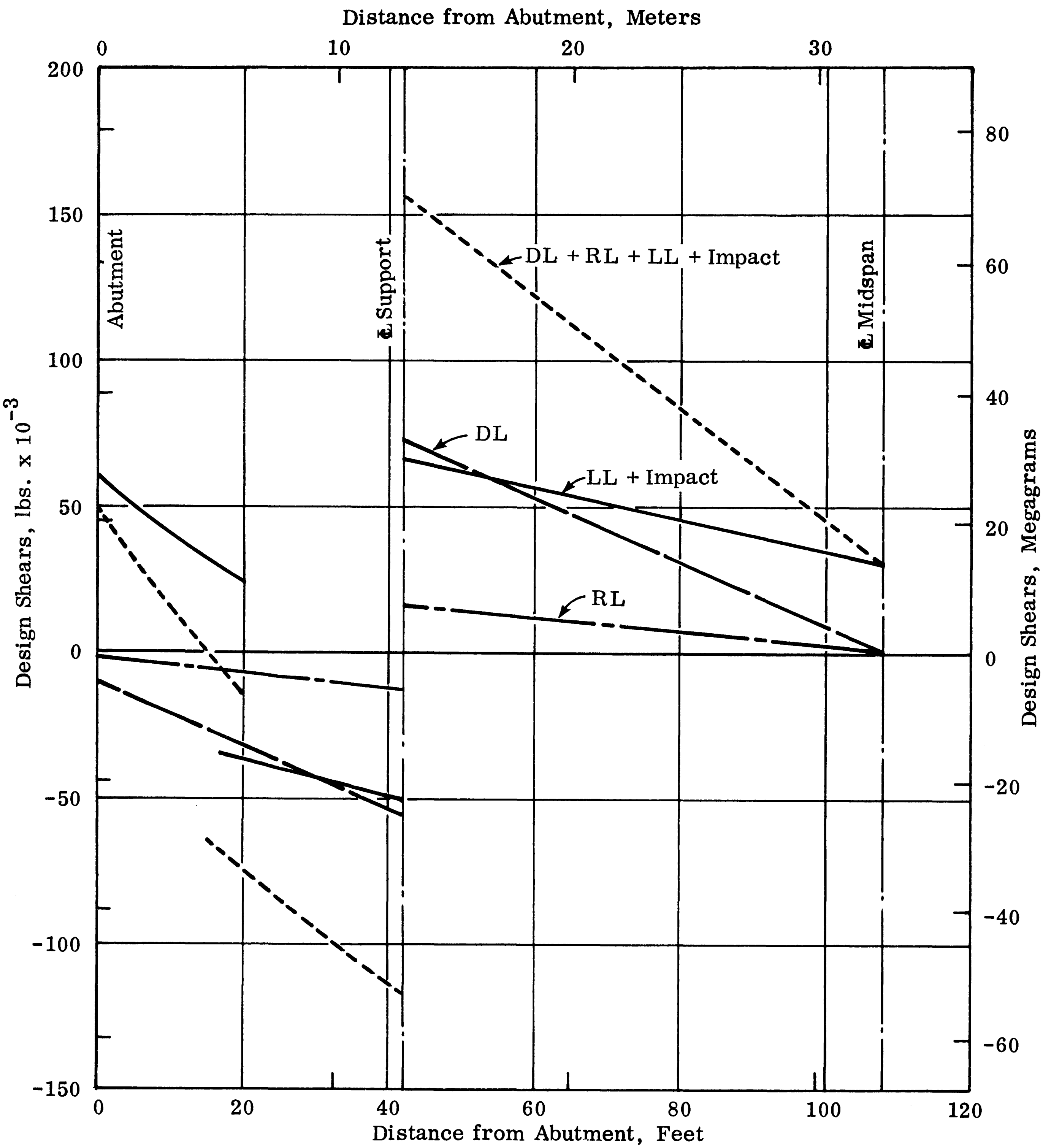

Figure A-6. Shear curves for continuous span

APPENDIX B

Computer Program and Sample Loading Data used for the Design of Plan D

532

```
      PROGRAM SPPINK(INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
      DIMENSION F(202),D(202),SL(20503),WM(202)
      DIMENSION JDF(93,3),O(93,6)
      DIMENSION E(265),A(265),T(265),X(265),Y(265),Z(265)
      DIMENSION B(14,5),W(265,3)
      REAL L(265),K(6,6)
      INTEGER R(265,2),SNO
C     MS=(M/2)*(M+1)
      MS=20503
      READ(5,100) M,M3,N
  100 FORMAT(3I5)
      M9=3*M3-M
      DO 1 I=1,M3
      DO 1 J=1,3
    1 JDF(I,J)=1
      DO 2 I=1,M9
      READ(5,101)JN,MD
    2 JDF(JN,MD)=0
  101 FORMAT(2I5)
      DO 3 K1=1,N
    3 A(K1)=1.
      DO 4 I=1,M3
    4 READ(5,200)(O(I,J),J=1,6)
  200 FORMAT(6F10.2)
      IC=1
      DO 5 I=1,M3
      DO 6 J=1,3
      IF(JDF(I,J).LT.1) GO TO 7
      JDF(I,J)=IC
      F(IC)=O(I,J+3)
      IC=IC+1
    7 CONTINUE
    6 CONTINUE
    5 CONTINUE
      IF(M.EQ.IC-1) GO TO 88
      GO TO 995
   88 CONTINUE
      DO 9 I=1,N
      READ(5,101)R(I,1),R(I,2)
      JN1=R(I,1)
      JN2=R(I,2)
      X3=O(JN2,1)-O(JN1,1)
      Y3=O(JN2,2)-O(JN1,2)
      Z3=O(JN2,3)-O(JN1,3)
      L1=SQRT((X3**2)+(Y3**2)+(Z3**2))
      X(I)=X3/L1
      Y(I)=Y3/L1
       Z(I)=Z3/L1
    9 L(I)=L1
      READ(5,100)IC9,IR9,NCM
      IT9=IC9+IR9
      DO 11 I=1,IT9
   11 READ(5,220)(B(I,J),J=1,5)
  220 FORMAT(5F10.4)
      DO 68 I=1,M
   68 WM(I)=-0.35
      ICI=0

  150 CONTINUE
      ICI=ICI+1
      DO 12 I=1,MS
   12 SL(I)=0.
      DO 13 I=1,N
      R1=A(I)/L(I)
      K(1,1)=K(4,4)=R1*X(I)**2
      K(1,4)=K(4,1)=-K(1,1)
      K(2,2)=K(5,5)=R1*Y(I)**2
      K(2,5)=K(5,2)=-K(2,2)
      K(1,2)=K(2,1)=K(4,5)=K(5,4)=R1*X(I)*Y(I)
      K(1,5)=K(2,4)=K(4,2)=K(5,1)=-K(1,2)
      K(3,3)=K(6,6)=R1*Z(I)**2
      K(3,6)=K(6,3)=-K(3,3)
      K(2,3)=K(3,2)=K(5,6)=K(6,5)=R1*Y(I)*Z(I)
      K(2,6)=K(6,2)=K(3,5)=K(5,3)=-K(2,3)
      K(1,3)=K(3,1)=K(4,6)=K(6,4)=R1*X(I)*Z(I)
      K(1,6)=K(6,1)=K(3,4)=K(4,3)=-K(1,3)
      J5=0
      DO 14 M1=1,2
      DO 15 J=1,3
      J5=J5+1
      JN1=R(I,M1)
      IF (JDF(JN1,J).EQ.0) GO TO 15
      I1=JDF(JN1,J)
      J6=0
      DO 17 M2=1,2
      DO 18 J1=1,3
      J6=J6+1
      JN2=R(I,M2)
      IF (JDF(JN2,J1).EQ.0) GO TO 18
      J2=JDF(JN2,J1)
      IF(J2.LT.I1) GO TO 18
      LN=(2*(M*(I1-1)+J2)+I1*(1-I1))/2
      SL(LN)=SL(LN)+K(J5,J6)
   18 CONTINUE
   17 CONTINUE
   15 CONTINUE
   14 CONTINUE
   13 CONTINUE
      CALL SYMINV(SL,M,MS,IFAIL)
      DO 19  I=1,M
   19 D(I)=0.
      DO 20 I=1,M
      LN=(2*M*(I-1)+I*(3-I))/2
      D(I)=SL(LN)*(F(I)+WM(I))+D(I)
      IF(I.EQ.M) GO TO 20
      IP1=I+1
      DO 21 J=IP1,M
      LN=LN+1
      D(J)=SL(LN)*(F(I)+WM(I))+D(J)
   21 D(I)=SL(LN)*(F(J)+WM(J))+D(I)
   20 CONTINUE
      DO 25 I=1,N
   25 E(I)=0.
      DO 27 I=1,M
   27 WM(I)=0.

      DO 38 I=1,M
      JN1=R(I,1)
      JN2=R(I,2)
      IF (JDF(JN1,1).EQ.0) GO TO 31
      SND=JDF(JN1,1)
      E(I)=E(I)-D(SND)*X(I)
   31 IF (JDF(JN2,1).EQ.0) GO TO 32
      SND=JDF(JN2,1)
      E(I)=E(I)+D(SND)*X(I)
   32 IF (JDF(JN1,2).EQ.0) GO TO 33
      SND=JDF(JN1,2)
      E(I)=E(I)-D(SND)*Y(I)
   33 IF (JDF(JN2,2).EQ.0) GO TO 34
      SND=JDF(JN2,2)
      E(I)=E(I)+D(SND)*Y(I)
   34 IF (JDF(JN1,3).EQ.0) GO TO 35
      SND=JDF(JN1,3)
      E(I)=E(I)-D(SND)*Z(I)
   35 IF (JDF(JN2,3).EQ.0) GO TO 30
      SND=JDF(JN2,3)
      E(I)=E(I)+D(SND)*Z(I)
   30 CONTINUE
      DO 38 I=1,N
   38 T(I)=A(I)*E(I)/L(I)
      IC5=0
      P1=9.8695877
      E9=30000.
      DO 50 I=1,N
      J=IC9
      IF(I.GT.NCM) GO TO 473
      J=0
  473 J=J+1
      IF(J.GT.IT9) GO TO 998
      IF(I.GT.NCM) GO TO 477
      IF(J.GT.IC9) GO TO 998
  477 IF (T(I).GT.0.) GO TO 510
C     REM:COMPRESSION MEMBER
      C3=SQRT(2.*P1*E9/B(J,5))
      CK3=1.*L(I)/B(J,2)
      IF(I.LE.223) GO TO 478
      CK3=CK3/1.246
  478 IF (CK3.GT.C3) GO TO 490
      F5=(1.-CK3**2/(2.*C3**2))*B(J,5)
      F5=F5/((5./3.)+(3.*CK3/(8.*C3))-(CK3**3/(8.*C3**3)))
      GO TO 495
  490 IF (CK3.GT.200.) GO TO 473
      F5=12.*P1*E9/(23.*CK3**2)
  495 A1=-T(I)/F5
      IF (B(J,1).LT.A1) GO TO 473
      A1=B(J,1)
      GO TO 530
C     REM:TENSION MEMBER
  510 A1=T(I)/(B(J,5)*.6)
  513 IF (B(J,1).LT.A1) GO TO 525
      A1=B(J,1)
      GO TO 530
  525 J=J+1

      IF (J.GT.IT9) GO TO 998
      IF(I.GT.NCM) GO TO 510
      IF(J.GT.IC9) GO TO 998
      GO TO 513
  530 IF (ABS(A(I)-A1).GT..1) GO TO 532
      IC5=IC5+1
  532 W(I,1)=B(J,3)
      W(I,2)=B(J,4)
      W(I,3)=B(J,5)
      W1=W(I,1)*L(I)/24000.
      J1=R(I,1)
      J2=R(I,2)
      L1=JDF(J1,2)
      L2=JDF(J2,2)
      IF(L1.EQ.0) GO TO 47
      WM(L1)=WM(L1)-W1
   47 IF(L2.EQ.0) GO TO 50
      WM(L2)=WM(L2)-W1
   50 A(I)=A1
      WRITE(6,550)
  550 FORMAT(1H1)
      WRITE(6,562)
  562 FORMAT(10X,16HDEFLECTIONS(IN.),/)
      DO 44 I=1,M
      DEL=D(I)/30000000.
   44 WRITE(6,564)DEL
  564 FORMAT(10X,F11.8)
      WRITE(6,550)
      WRITE(6,566)
  566 FORMAT(26X,10HSTEEL PIPE,//,
     115X,31H3 IN. NOMINAL DIA.  FY = 36 KSI,//)
      WRITE(6,565)
  565 FORMAT(2X,31HMEMBER NO.  WALL THICKNESS(IN.),3X,
     128HAREA(INSQ.)       FORCE(KIPS),//)
      W9=0.
      DO 40 I=1,NCM
      WRITE(6,570)I,W(I,2),A(I),T(I)
   40 W9=W9+W(I,1)*L(I)/12.
  570 FORMAT(3X,I5,5X,F10.3,10X,F10.2,10X,F10.4)
      WRITE(6,550)
      WRITE(6,567)
  567 FORMAT(15X,29HRECTANGULAR STRUCTURAL TUBING,//,
     116X,26H8 IN. X 6 IN. NOMINAL SIZE,//)
      WRITE(6,568)
  568 FORMAT(2X,31HMEMBER NO.  WALL THICKNESS(IN.),3X,
     153HAREA(INSQ.)       FORCE(KIPS)        YIELD STRENGTH(KSI),//)
      NP1=NCM+1
      DO 42 I=NP1,N
      WRITE(6,572)I,W(I,2),A(I),T(I),W(I,3)
   42 W9=W9+W(I,1)*L(I)/12.
  572 FORMAT(3X,I5,5X,F10.3,10X,F10.2,10X,F10.4,10X,F10.0)
      WRITE(6,575)W9
  575 FORMAT(/,2X,32HTOTAL WEIGHT OF STEEL IN LBS =   ,F10.2)
      IF (IC5.EQ.N) GO TO 999
      IF(ICI.GE.3) GO TO 999
      GO TO 150
  995 WRITE(6,600)

  600 FORMAT(2X,29HNUMBER OF DOF NOT ESTABLISHED)
  997 GO TO 999
  998 WRITE(6,602)I
  602 FORMAT(2X,37HNEED LARGER SECTION FOR MEMBER NUMBER,2X,I5)
  999 CONTINUE
      END
```

Figure B-1. Fortran computer program used to design space frame.

Figure B-1 shows the Fortran computer program which was developed to design the space frame. Sample calculations of the equivalent concentrated loads which were input into the computer to represent the various loading conditions follow Figure B-2, which provides an illustration of the application of load to a typical transverse section.

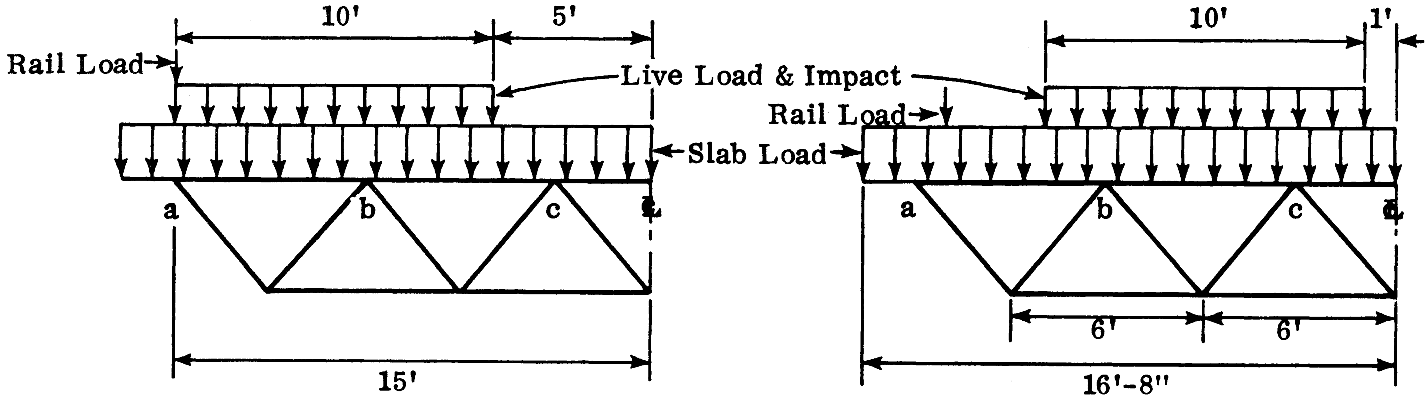

Figure B-2. Application of load to a transverse section of the space frame
(a) Case 1 loading (b) Case 2 loading

Basic conversion 1' = 0.305 m

<u>Sample Calculations</u>
(Basic conversion 1 kip = 4.448 kN)

Dead load for slab = 1.25 kips/ft. (18.24 kN/m)
(1.25 kips/ft.) x (L) = (1.125) x (7.67) = 9.59 kips/transverse section
Slab load on node (a) = (4.67/16.67) x (9.59) = 2.69 kips
Slab load on node (b) and (c) = (6/16.67) x (9.59) = 3.45 kips

Rail load on node (a) = (0.512 kips/ft.) (7.67) = 3.93 kips

Live load and impact for AASHTO uniform lane load distributed transversely according to the loading case are as follows:

I = Impact

end span	I = 30.0%
center span	I = 19.58%
average span	I = 23.67%

LL + I = (0.640) (7.67') (1.30) = 6.38 kips (positive moment end span)

LL + I = (0.640) (7.67') (1.1958) = 5.87 kips (positive moment center span)

LL + I = (0.640) (7.67') (1.2365) = 6.07 kips (negative moment)

The concentrated joint loads shown below are equivalent to the distributed LL + I for maximum negative moment.

Case 1	Case 2
Node (a) = (6.07)(3/10) = 1.82 kips	Node (a) = 0.0 kips
Node (b) = (6.07)(6/10) = 3.64 kips	Node (b) = (6.07)(5/10) = 3.04 kips
Node (c) = (6.07)(1/10) = 0.61 kips	Node (c) = (6.07)(5/10) = 3.03 kips

Total joint loads for the maximum negative moment loading condition are as follows.

$$TL = (LL + I) + DL + RL$$

Case 1	Case 2
Node (a) = 8.44 kips	Node (a) = 6.62 kips
Node (b) = 7.09 kips	Node (b) = 6.48 kips
Node (c) = 4.06 kips	Node (c) = 6.48 kips

Live load and impact for AASHTO concentrated loads distributed transversely where required to produce maximum moments and shears are shown below.

(18 kips) (1.2365) = 22.26 kips	(max. negative moment)
(18 kips) (1.1958) = 21.52 kips	(max. positive moment center span)
(18 kips) (1.3) = 23.40 kips	(max. positive moment end span)
(26 kips) (1.2365) = 32.15 kips	(center span shear)
(26 kips) (1.3) = 33.80 kips	(end span shear)

Additional joint loads resulting from the application of the AASHTO 18 kip concentrated loads for maximum negative moment are as follows.

Case 1	Case 2
Node (a) = (22.26)(3/10) = 6.68 kips	Node (a) = 0.0 kips
Node (b) = (22.26)(6/10) = 13.36 kips	Node (b) = (5/10)(22.26) = 11.13 kips
Node (c) = (22.26)(1/10) = 2.23 kips	Node (c) = (5/10)(22.26) = 11.13 kips

One-half of the tabulated values were used on the nodes at each end of the space frame.

APPENDIX C

FABRICATORS AND CONTRACTORS PROVIDING COST ESTIMATES

Associated Steel Products, Inc.
Charlottesville, Virginia

Bristol Steel & Iron Works, Inc.
Richmond, Virginia

J. Lawson Jones Construction Company, Inc.
Clarksville, Virginia

Montague Betts Company
Lynchburg, Virginia

Roanoke Iron & Bridge Works, Inc.
Roanoke, Virginia

Wiley N. Jackson Company, Inc.
Roanoke, Virginia