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STRUCTURAL SYSTEMS IN SUPPORT OF SAFETY: NEW HIGHWAY STRUCTURES DESIGN CONCEPTS. VOLUME II. PRELIMINARY DESIGNS AND ENGINEERING DATA

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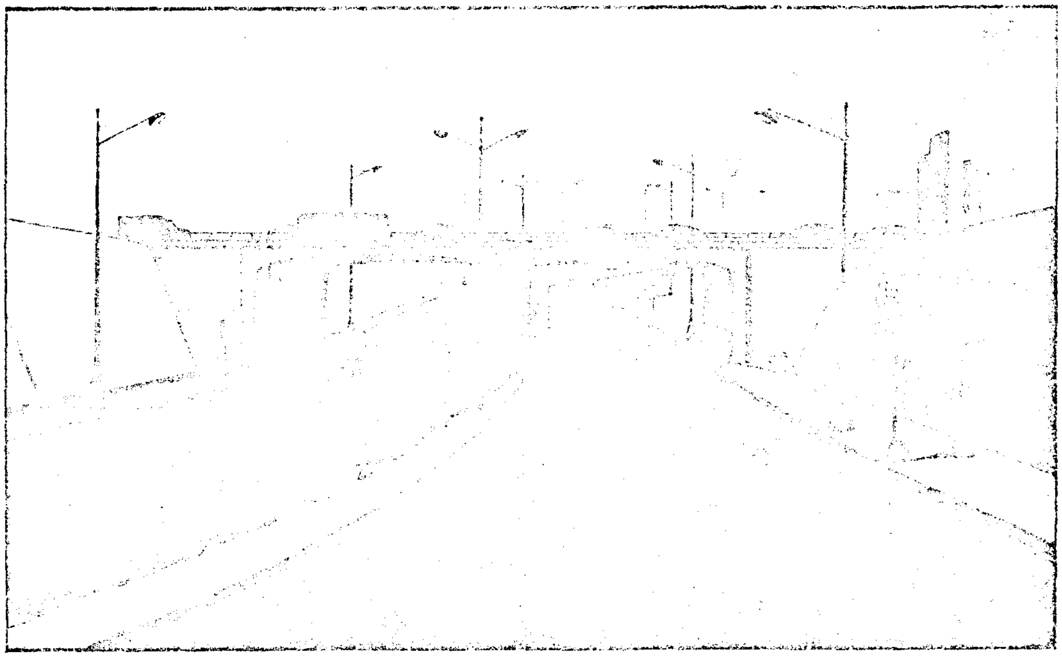
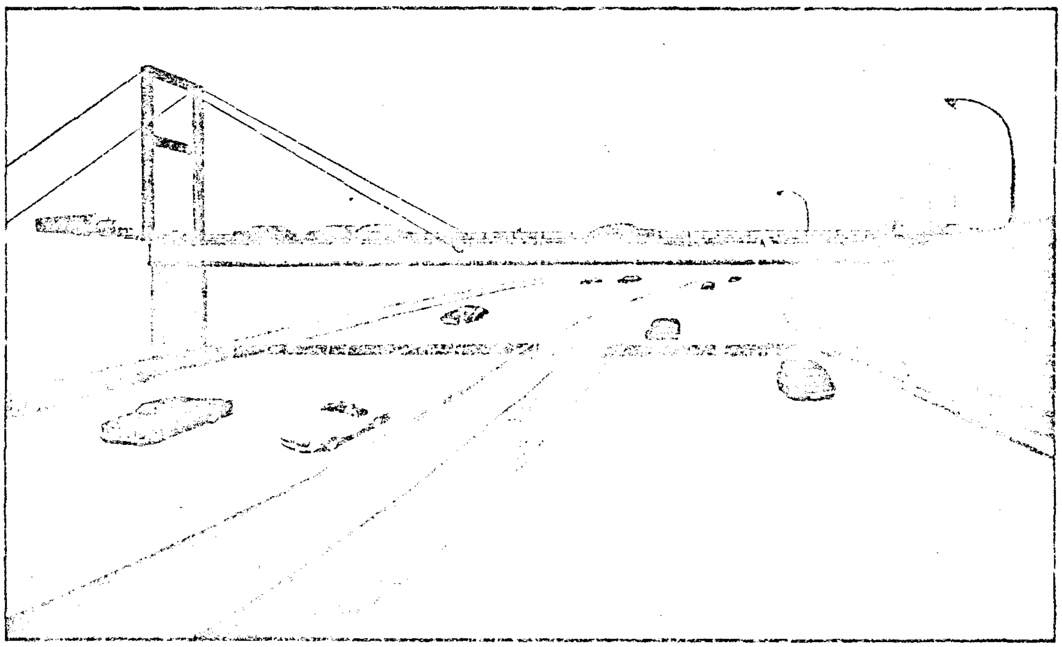
September 1969





NEW STRUCTURES CONCEPTS FOR HIGHWAY SAFETY

VOLUME II. PRELIMINARY DESIGNS AND ENGINEERING DATA

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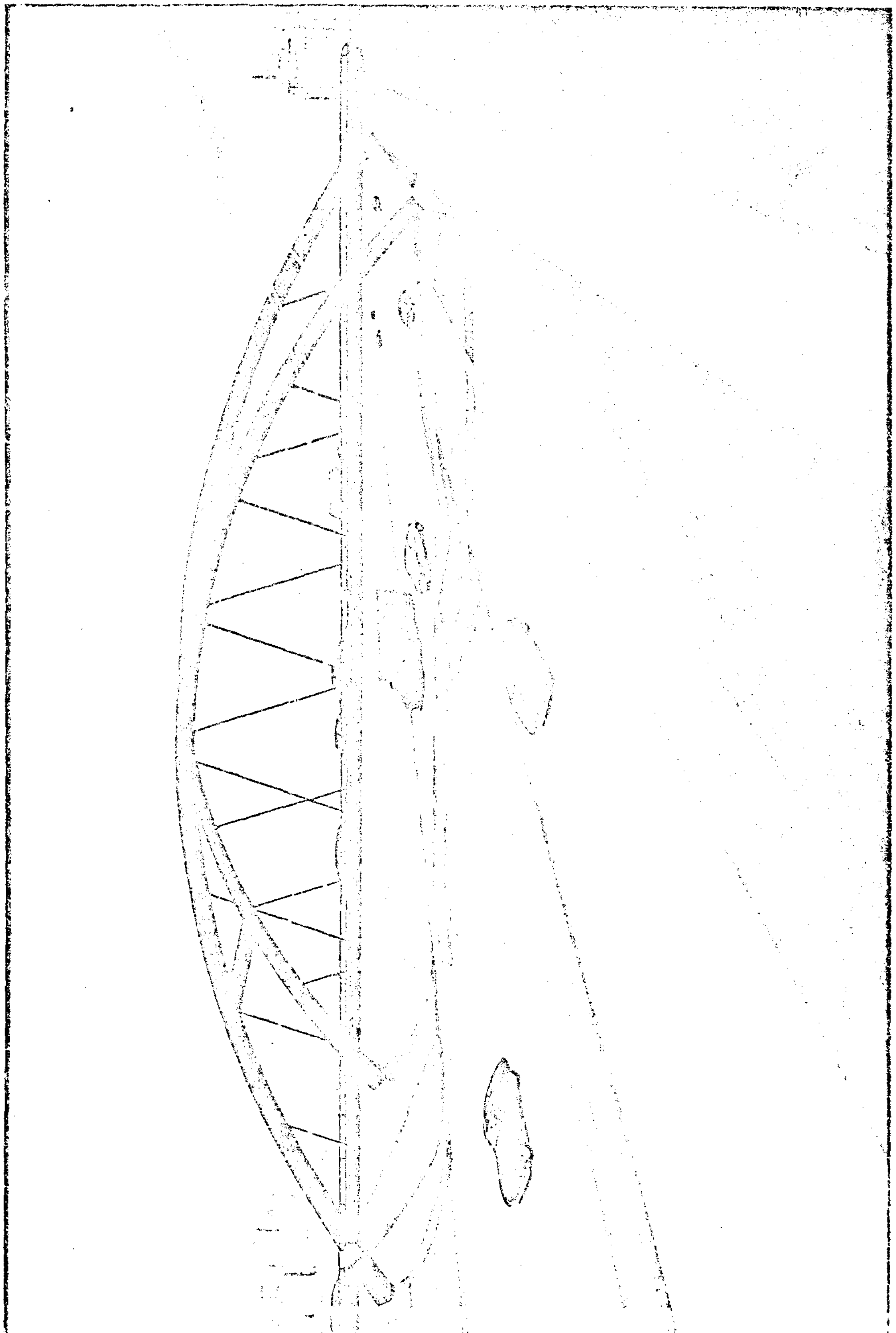

STRUCTURAL SYSTEMS IN SUPPORT OF SAFETY: NEW HIGHWAY STRUCTURES DESIGN CONCEPTS

FINAL REPORT
SwRI Project No. 03-2173

VOLUME II. PRELIMINARY DESIGNS AND ENGINEERING DATA

Prepared under
Contract FH-11-6638
for
The Bureau of Public Roads
Federal Highway Administration
Department of Transportation

September 1969

FOREWORD

The investigation reported herein was conducted by Southwest Research Institute in the Department of Structural Research. Joseph E. Minor and Maurice F. Bronstad served as the project Principal Investigators. This report was prepared under Contract No. FH-11-6638 with the Bureau of Public Roads, Federal Highway Administration, Department of Transportation. The scope of work required development of imaginative concepts for highway structures which are responsive to new safety requirements. It was specified, however, that these concepts be limited to structural schemes employing cable systems in applications which differ from those used in conventional suspension bridges.

The report is presented in three separate volumes:

- Volume I - Research Information
- Volume II - Preliminary Designs and Engineering Data
- Volume III - Supporting Data

Each volume is responsive to different information requirements and is essentially complete within itself. For example, those concerned with study methodology and concept development will be interested in Volume I, while practicing engineers responsible for implementation will find that the concise information presented in Volume II is more applicable. Individuals in both categories who wish to pursue their interests in more detail will find the supporting data contained in Volume III useful.

Volume II contains preliminary designs and supporting data for bridge, sign, and lighting system support concepts, which were selected as feasible cable-supported structural applications that are responsive to new, safety-related geometric design criteria.

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ACKNOWLEDGMENTS

The assistance and information provided by many persons in state highway departments and the Bureau of Public Roads are gratefully acknowledged. Direct contributions in the concept selection and review process by Mr. F. J. Tamanini and Mr. Lloyd Caves of the Bureau of Public Roads, Office of Research and Development, are acknowledged. Suggestions and information provided by Messrs. Odis H. Clark, Jr., Leroy A. Surtees, Jr., and Malcolm L. Steinberg of the Texas Highway Department are also acknowledged. The contribution made by Mr. Warren Hunter in presenting artist's renderings of the bridge concepts is also acknowledged. Direct assistance provided by Institute staff members is noted as follows: Dr. Thein Wah, who formulated the theoretical methods; Mr. Raymond E. Kirksey, who supervised computer programming and computer operation activities; and Messrs. Leonard U. Rastrelli, Robert Sherman, Norman I. Turner, and R. Kirk Gregory, who contributed concepts and ideas.

ABSTRACT

Volume II of this report contains results of the program presented as artist's sketches, tabulations of engineering data, preliminary design drawings, design assumptions and criteria, and cost estimates for the fabrication of prototype structures, and subsequent procurement in quantity. Three new bridge concepts which employ cable supports are advanced as feasible structural schemes for effecting the removal of massive support structures from the area adjacent to the roadway. Two of the concepts are advanced as feasible structures for new bridge applications; these are the Leaning Arches Bridge and the Bridle Bridge. The Leaning Arches Bridge and the remaining concept, the Frame Bridge, are advanced as feasible structures for use in modified existing bridge applications to permit removal of hazardous interior bents and abutments. Cable-supported structures concepts for highway signs and lighting systems are also presented.

Preliminary designs are developed, fo the feasible concepts identified above, in response to specific design situations which represent the severe requirements dictated by new, safety-related design criteria. The preliminary designs and engineering data presented are quantitative insofar as they respond to the specific geometric and design situation selected. The detailed nature of the design and cost data presented in this manner will provide the highway engineer with both general and specific appraisals of the applications of these concepts to highway practice.

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I. INTRODUCTION

The Structural Systems in Support of Highway Safety (4S) program sponsored by the Bureau of Public Roads is a short range, quick payoff research endeavor designed to reduce the severity of single vehicle accidents on the Nation's highways. The objectives of this accelerated program may be summarized as: (1) to develop structural systems concepts for the elimination of rigid obstacles and other obstructions along the highways and (2) to develop devices and structural arrangements for vehicle impact attenuation, deflection or entrapment to assure that collisions with these devices will be of minimum severity. An examination of the statistics on single vehicle collisions confirmed the advisability of pursuing both approaches in achieving a solution to the problem.

The scope of work for the project summarized herein was responsive to the former objective and required that new, imaginative and creative concepts for highway structures be developed. It was specified, however, that these concepts be limited to structural schemes employing structural cable systems in applications which differ from those used in conventional suspension bridges. The development of unique concepts involved a trial and error approach. Activities were primarily concerned with employing the creative capabilities of a project team composed of specialists in architecture, aerospace structures, civil engineering structures, and material development. Ideas were envisioned, formulated, translated into sketches, and subjected to an evolution procedure wherein conceptual drawings were repeatedly reworked. Each stage of the evolution procedure represented the originator's attempt to effect the compromises needed to transform an idealized structure into one with a reasonable degree of implementation feasibility. Concepts that survived an individual's evaluations were then subjected to a critical, detailed appraisal by two or more of the project's team members. At this stage, many of the newly conceived concepts were rejected and, therefore, eliminated from further consideration. For the most part, rejection was a reflection of technical or economic barriers related to current practices in highway bridge design and construction.

This volume of the three-volume research report contains specific results of the project presented in concise, summary form. Research activities* led to preliminary design and engineering data regarding several feasible, cable-supported structural systems that are responsive to new, safety-related geometric design criteria. Summaries of research considerations and general information regarding concepts applications are included as a preliminary section in this volume. This section is followed by presentations of design concepts for new bridge applications (Section III), design

*Summarized in Volume I (Research Information).

concepts for modifying existing bridges (Section IV), and new design concepts for sign and lighting system support structures (Section V). A summary and conclusions section is included as Section VI.

II. RESEARCH SUMMARY AND CONCEPTS APPLICATIONS DISCUSSION

A. Background

The time has passed when the highway designer can concern himself only with providing the motorist with a smoothly negotiable roadway having good riding qualities. Recent accident experience has shown that a large number of fatalities occur as a result of errant vehicles leaving the roadway and, subsequently, striking fixed objects. Therefore, in addition to riding quality considerations, emphasis is now being placed on providing the "errant" motorist with wide, unobstructed areas beyond the edges of the roadway in which to recover control of his vehicle. The February 1967 special report of the AASHO Traffic Safety Committee(1)* and current recommendations concerning placement of median barriers and guardrails(2) suggest that unobstructed areas 30 feet wide adjacent to both sides of the roadway are desirable. Accordingly, a minimum median width of 60 ft is necessary for pier placement in a median with no guardrail protection.

1. Research Summary

A specific 4-lane divided highway section which requires the maximum unobstructed horizontal clearance, as suggested by the new clearance criteria, was chosen for use as a standard in configuring structures for analysis and evaluation in this investigation. An overpass, sign, or lighting system structure employed with this roadway section must provide 170 feet of horizontal clearance (spanning a 60-foot-wide median, two 30-foot outside shoulder clearances and two 25-foot-wide pavements) in order to free the roadway of shoulder guardrail and median barriers. The geometries of structures considered in this investigation were governed by these clearance requirements, which are illustrated in Figure 1. It must be emphasized that the roadway section and clearance geometries are not presented as recommended design criteria, but were selected as demanding requirements for use in the research investigation to establish a basis for evaluating structural concepts.

Extensive research activities which included literature searches and conference-type concept identification and evaluation procedures† provided a basis for selection of three feasible cable-supported structural concepts for bridges and a feasible cable-supported structural concept for highway signs and lighting systems. Design methods, preliminary designs and cost estimates for new bridges, modifications of existing bridges and signs and

*Numbers in parentheses refer to List of References, Section VII.

†A comprehensive summary of research activities is included in Volume I (Research Information).

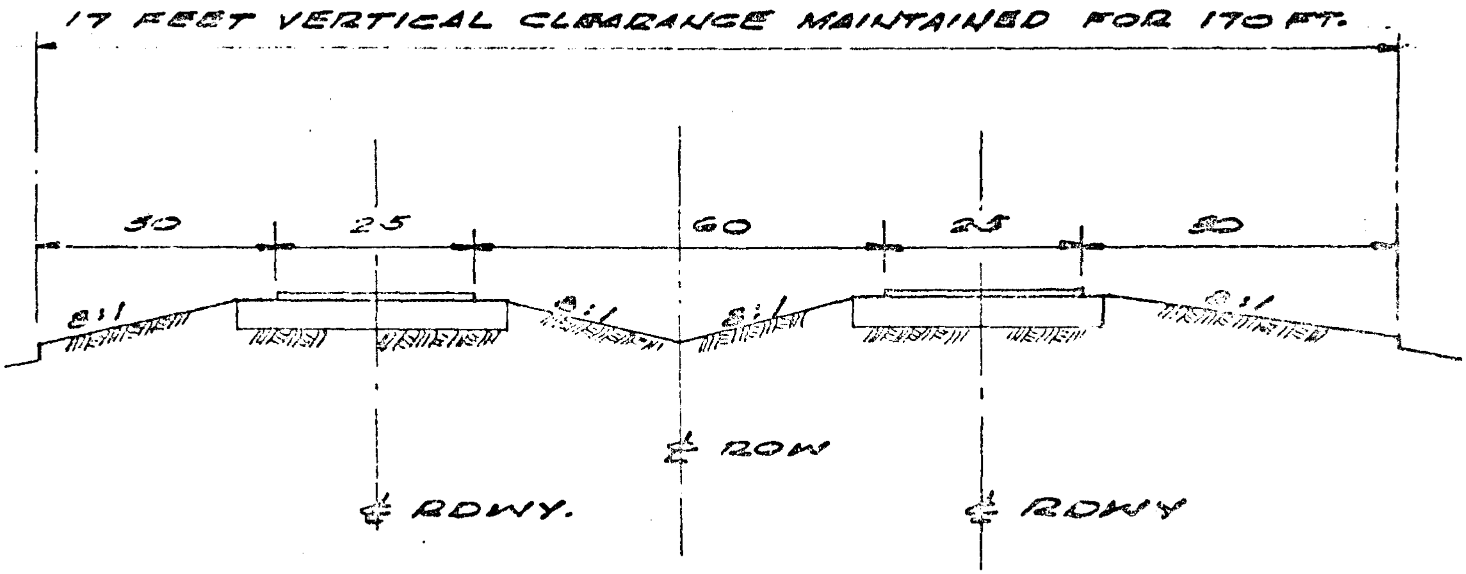


FIGURE 1. UNOBSTRUCTED CLEARANCE REQUIREMENTS FOR STRUCTURES CROSSING FOUR-LANE DIVIDED HIGHWAYS AS ESTABLISHED BY NEW SAFETY CONSIDERATIONS

lighting systems are presented in this volume, using these concepts and the forementioned geometric requirements to formulate basic structural schemes. The purpose of presenting outlines of design procedures, preliminary designs and cost estimates in this fashion is to provide highway engineers with specific, quantitative information for use in making rational decisions when considering structural concepts for new or replacement bridges, or other highway structures.

2. Concept Selection

The specific structural concepts presented in this report represent a departure from conventional structural design. Therefore, the lack of an "experience factor" will present problems to highway engineers faced with making decisions regarding use of new structural concepts of the type presented. In recognition of this aspect of the decision-making process, departures from conventional structural design and construction processes have been minimized in developing the preliminary designs, insofar as was possible. In this regard, it is noted that many current design techniques will continue to be applicable to the design of new structures.

Factors which will influence selection of a structural concept for a particular highway application include: (1) safety, (2) economics, and (3) aesthetics. By employing the uniform clearance requirements outlined in Figure 1, as a standard for analysis, each of the preliminary designs presented in this report has been made equal with regard to meeting safety requirements. Therefore, economics and aesthetics become principal factors when considering the merits of the specific systems presented. To assist in the evaluation and selection process, the weights of the primary structural members and the total bridge weight are included in the tabulations of engineering data as economics related comparison parameters. By employing these data as cost-related parameters, in conjunction with the preliminary cost estimates, general appraisals regarding the economy of one structural concept versus another in a particular application may be made. In utilizing the specific engineering data presented as an aid in the concept selection process, it must be revealed that (1) these preliminary designs and cost estimates were developed for specific geometric requirements and ideal site considerations (i.e., Figure 1 geometric requirements and 0° skew crossing structures) and (2) additional design and construction-related considerations will enter into economic evaluations regarding specific applications. A complete structural analysis (including dynamic analysis) and detailed economic studies will be necessary before final conclusions can be drawn concerning feasibility of any of the concepts presented in a given application.

3. Preliminary Designs

Because they represent distinctly different design and construction situations, the portion of the report devoted to presenting preliminary designs is separated into (1) New Bridge Concepts, (2) Concepts for Modifying Existing Bridges, and (3) Concepts for Supporting Signs and Lighting Systems. Three feasible bridging concepts are employed in presenting two preliminary designs for new bridges and two preliminary designs for modifying existing bridges. These concepts are the Leaning Arches Bridges, the Bridle Bridge, and the Frame Bridge. Three additional bridge concepts that were not selected for preliminary design consideration, but possess certain potentials for responding to safety-related geometric requirements, are presented as potentially feasible concepts for new and modified bridges. These additional concepts are the Stayed Girder Bridge, the Braced Arch Bridge, and the Leaning Piers Bridge. One structural concept is employed in presenting preliminary designs for sign and lighting system support structures.

B. Design Criteria for Bridges

A design process that is similar in procedural steps to current methods which a highway engineer employs was used in developing the preliminary designs for bridges presented in this investigation. As a first step in accomplishing preliminary design of a bridge, a decision is required regarding the desired horizontal and vertical clearance envelope. Figure 1 illustrates the envelope chosen for this study; all preliminary designs developed in this report provide the horizontal and vertical clearance illustrated. A 26-foot roadway with a 17-foot overhead clearance was chosen as a standard section representing the crossing roadway for purposes of the study. A 6-1/2-inch concrete slab with 9-inch curbs was used as a typical bridge deck. This specific section was chosen for analysis standardization purposes, and its use in this investigation should not restrict the application of the bridge concepts presented to bridges with this particular crossing roadway section.

The AASHO Standard Specifications for Highway Bridges was used in this investigation to develop design loads and determine allowable material stresses, although it is clear that current AASHO deflection and load criteria may not be appropriate for the design of cable-supported bridges. The uniqueness of these new structures, including their relative flexibility, may preclude effective design by a code oriented toward other types of bridge designs. The deflections of cable-supported structures may not meet (nor do they necessarily need to meet) current AASHO requirements. A simple dynamic loading could be used in a modified design procedure to achieve a proper design and provide the design engineer with the required confidence in the structure.

H20-S16 highway loadings were employed with conventional static analysis procedures to develop preliminary designs. The standard lane loading was used throughout as the design loading to permit simplification of the preliminary design procedure. Impact factors were computed and applied to live loads using the AASHO impact equation. For preliminary analysis and design purposes only, longitudinal forces, lateral forces, wind loads, thermal loads, etc., were not directly considered. Dead-loads were estimated to include the concrete bridge deck and were reduced to uniformly distributed loads for analysis purposes.

C. Design Criteria for Sign and Lighting System Supports

Establishment of criteria to govern design of overhead sign support structures and lighting system support structures, which are responsive to the objectives of this study, began with a review of current design standards. Current practice is represented by four AASHO documents(1, 3, 4, 5). As in the case of the bridge studies, criteria for the design of sign and lighting support structures may be divided into three areas: geometry, load, and aesthetics. With respect to geometry, the AASHO specification (3) states that "... it is advisable to provide greater vertical clearance for sign bridges (than for bridge structures) ...." With this requirement in mind, geometric requirements for sign and lighting system support structures were established as shown in Figure 1, with the exception that the required vertical clearance must be 18 feet over a width of 170 feet, rather than the 17 feet noted for bridge structures. Other geometric considerations pertain to the horizontal clearances required for exit and entrance ramps, and near access roads (e.g., sign structures supports should not be placed in a "gore" area). Since the geometric configurations of these structures are sensitive to characteristics of a given site, it was deemed advisable to adopt a representative horizontal clearance as shown in Figure 1 for the purpose of developing preliminary designs.

Load requirements for overhead sign supports are detailed in the "AASHO Specifications for the Design and Construction of Structural Supports for Highway Signs"(3); these requirements include dead, live, ice, and wind loads.* The first two involve structure weight and walkway forces, respectively; the latter two are concerned with forces which vary according to geographic area, and require detailed analysis based on ice weight and wind pressures. For the purpose of developing preliminary designs of sign supports, these load conditions were simplified into a representative load requirement. This representative load consists of an estimated combined

*Ross and Olson (6) have concluded that this statically applied load is unconservative and that new design criteria should be developed for highway signs.

dead load and ice load, and a wind load computed using a wind pressure of 55 psf, suitably modified by supplementary factors for application to structural members. In addition, the wind loads were considered to act normal to the vertical face of the sign and support.

The aspect of aesthetics is covered in current standards by general statements and guidelines. In the AASHO specification, for example, it is noted that "Within the limits of practical economics and with primary regard for the utility function performed by overhead sign supports, features which promote the aesthetics of such structures should receive proper attention . . . ." A specific guideline states "Aesthetics will be improved if the upper and lower edges of two or more sign panels on a single overhead sign structure produce parallel horizontal lines." As in the case of bridge concepts evaluations, sign structure aesthetics were given a qualitative role in the appraisals of concept designs.

The geometric requirements for present day lighting systems are based on roadway illumination and uniformity requirements which are provided by conventional 400- to 1000-watt mercury vapor lamps positioned 30 to 60 feet high above the edge of the roadway at intervals ranging from 150 to 350 feet. Load design criteria for lighting system support structures include: (1) the dead load of the lamp, plus its support superstructure, (2) the dead load due to ice, and (3) wind live load, as applicable for the geographic area of concern. Deflection criteria for light supports are not as restrictive as for sign supports and generally allow deflection up to 10 percent of the support length for aluminum and 5 percent for steel. Materials criteria embody stress allowables and weatherability. Materials selection embodies evaluation of many factors, including site conditions.

III. PRELIMINARY DESIGNS FOR NEW BRIDGES

Two structural design concepts employed as new bridge structures were selected for analysis and preliminary design: the Leaning Arches Bridge and the Bridle Bridge. The gross geometries of structures employed in the analysis and preliminary design discussed herein were dictated by the geometry and clearance requirements of both the crossing and crossed roadway. These requirements are defined in Section II. Although preliminary designs are presented in detail, it should be emphasized that prior to implementing the designs in specific applications dynamic analyses of the structural systems will be necessary to provide confidence in the structures from a dynamic stability standpoint. Tabulations of forces, moments, and shears used in developing the preliminary designs, as well as preliminary cost estimates for initial prototypes and subsequent procurement in quantity, are provided in tables and figures identified, with each concept application, in the following paragraphs. Additional, more detailed supporting data are included in Volume III.

A. Leaning Arches Bridge Concept

The arch has provided the engineer with an aesthetically pleasing structural concept since the beginning of engineered bridge construction more than 2000 years ago. The Leaning Arches Bridge, depicted by an artist in Figure 2, employs two arches which straddle the crossing roadway as they lean inward and join. The geometry of the leaning arches scheme provides both the crossed and crossing roadways with adequate vertical and horizontal clearances. Cables in the plane of the arches support transverse floor beams which, in turn, provide support of the floor system. A concrete slab and railing system complete the bridge deck.

1. Application Discussion

The Leaning Arches Bridge may prove to be particularly effective at bridge sites which provide ideal conditions for supporting the ends of the arch, although use of this concept need not be restricted to roadway cuts. In applying the leaning arch concept to specific bridge situations, the designer has considerable flexibility in selection of arch geometry, including the shape of the arch, the angle of inclination, and the number of cables supporting the floor system. Although the continuous arch presented here is parabolic and fixed at both ends, the designer has options available in selecting other schemes to fit particular site conditions (e. g., two hinged and three hinged arches, circular or elliptical in shape). These factors can be "tailored" to best suit the roadway geometries and site conditions under consideration.

2. Analysis Discussion

The arches chosen for analysis and preliminary design as a part of the Leaning Arches Bridge are parabolic with fixed ends as shown isometrically in Figure 3. Several methods of analysis may be employed to analyze an arch structure of this type(7, 8). The method of analysis outlined in Appendix B* for the leaning arches bridges is particularly suitable for computer application and, with the inclusion of AASHO specification loading, a complete spectrum of design values (i. e., design moments, shears, and deflections) can be generated for use in final design studies. In accomplishing preliminary analysis and design of the structure configured as shown in Figure 3, however, a less complex, more conventional approach was employed. The elastic center method was used to determine influence lines for the arch(7). After influence lines were determined for the specific, constant section parabolic arches, several structural schemes involving various cable configurations were considered. The cable support scheme illustrated in Figure 3 was selected as the most effective for the span and bridge geometry being considered.

An arch deflection theory presented by Borg and Gennaro(7) includes a criterion which, if met, precludes the necessity of accomplishing detailed investigations involving arch deflections as a part of the preliminary design process. Arches can be analyzed without deflection considerations if the factor of safety against buckling defined by

$$F.S. = \frac{N_{cr}}{N}\left[\frac{4\pi^2 EI}{\dfrac{L^2}{N}}\right] \qquad (1)$$

is greater than 3. In equation (1) L is the length of a two-hinged arch (L is approximately 0.7 times the arch length if the arch ends are fixed) and N is the maximum thrust in the arch section computed using appropriate arch analysis equations(7, 8).

Calculations employed in developing influence lines for the Leaning Arches Bridge are contained in Appendix D. The influence lines themselves are presented in Figure 4. Influence lines for the arches and cables are included. Applicable coefficients used in designing the continuous, equal span horizontal girders are available in many references. For the specific structure chosen for analysis, the floor system was considered to consist of eight continuous spans supported, for preliminary analysis and design purposes, by nondeflecting cables. It is recognized that actual cable elongations and arch deflections will have an influence on the bending moments

*In Volume III.

in the continuous beams (beam on elastic support effects); however, the effects of neglecting these elongations were not considered to be of significance in developing the preliminary design. This decision is supported by considerations of Equation (1) as applied to the specific structure analyzed.

Using the influence lines in Figure 4 to position live loads on the structure to produce maximum bending effects revealed that the maximum bending moment in the arch occurs at the third cable support point. Actual design values for each component of the structure, calculated by employing influence lines and moment coefficient tables to locate load positions providing maximum effects, are presented in Table I.

3. Preliminary Design Discussion

A preliminary design for the Leaning Arches Bridge was developed using the design values presented in Table I. The design calculations are summarized with the preliminary design in Table II. Although a steel structure was chosen for this specific preliminary design, other materials, such as concrete or aluminum, could be considered. A box section was selected for the arch for structural, as well as aesthetic, reasons. A plate girder section or other structural shapes could be considered and might prove more economical in certain situations. The angle of inclination of the arches chosen for purposes of illustrating this bridge concept results in relatively long transverse floor beams. Alternate geometric configurations with smaller angles of inclination would require separation of the arches at their crowns. The weight of additional structure required in achieving this configuration would probably offset the weight saved by shortening the floor beams. As another geometric alternative, the total height of the arch could be reduced by considering other geometric schemes. A thorough study of these and other geometric possibilities should be made to determine the desirable bridge configuration for a specific site condition.

The preliminary design does not include specific definitions of lateral bracing members and other appurtenances, although provision for miscellaneous steel is included in the design and cost estimates. Since lateral stability is not furnished by the cables, lateral bracing of the lower stringer flanges will be required (the concrete deck provides upper flange support). Abutment design will also require consideration of the several types of lateral forces. Furthermore, consideration of possible bridge rail deflection under the action of impacting vehicles will be necessary to assure that adequate clearance has been provided between the cables and roadway.

A preliminary design drawing of the Leaning Arches Bridge is presented in Figure 5. The weight of this structure, computed in Table II, is based on the member sizes shown in this figure. Members were designed for maximum design values noted in Table I. Although attempts were made to select the optimum section designs based on indicated design values, extensive design optimization procedures were not undertaken in developing this preliminary design.

4. Cost Discussion

Preliminary cost estimates for a prototype Leaning Arches Bridge (superstructure only) were prepared and are summarized in Table III. Unit prices were based on current* Texas Highway Department bid averages, with the exception of unit prices used for the arch structures and cables. Since the fabrication and erection of the arches described in the preliminary design are unconventional processes, when compared to current bridge construction practices, an inexperience factor was applied to the prototype (first structure) arch unit price. As experience is gained through construction of several structures of this type, the unit price for this item should approach that of conventional, fabricated steel items. The conventional, fabricated structure unit price is shown in parentheses in Table III. Unit prices for cables in the prototype structure were estimated, and a reduction in price for procurement and installation of this item in quantity is anticipated, if many structures are constructed. The reduced unit price is shown in parentheses adjacent to the prototype unit price item in Table III.

The estimated cost of the superstructure is based on the preliminary design shown in Table II and Figure 5. This design was qualified as being incomplete with respect to completion of dynamic analysis considerations and incorporation of other loading conditions. However, it is felt that the compensating generous allowance for miscellaneous material and potential material savings that may be experienced through design optimization efforts will permit the preliminary cost estimate to be employed as a first-order appraisal of actual superstructure costs.

*First quarter, calendar year 1969.

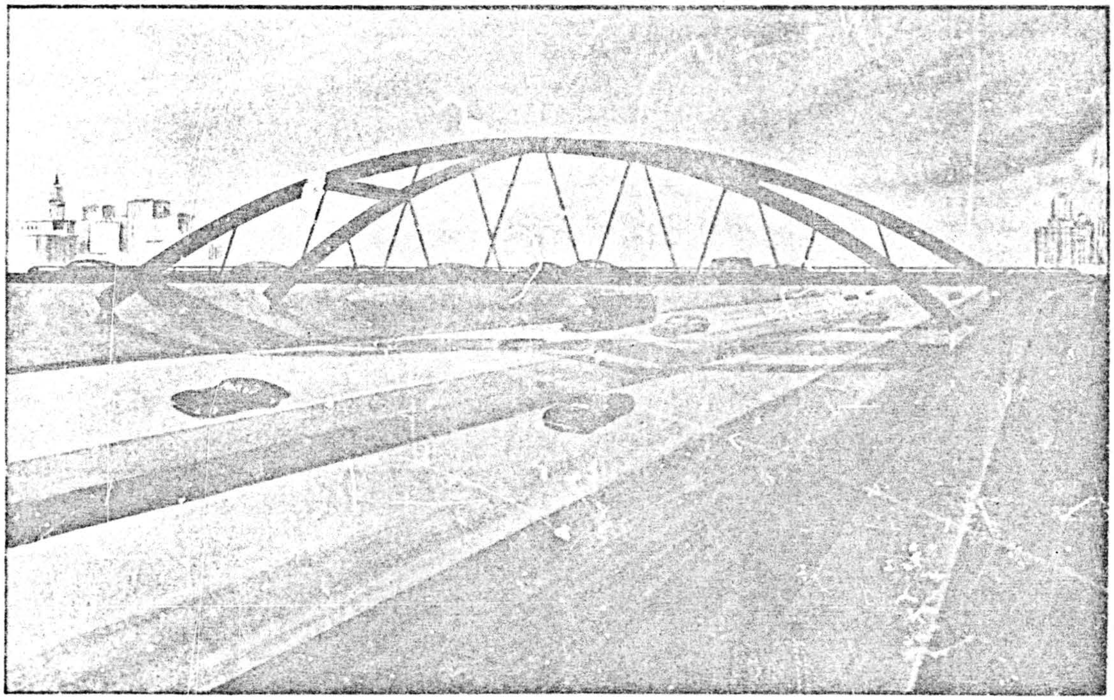


FIGURE 2. ARTIST'S SKETCH OF LEANING ARCHES BRIDGE CONCEPT IN NEW BRIDGE APPLICATION

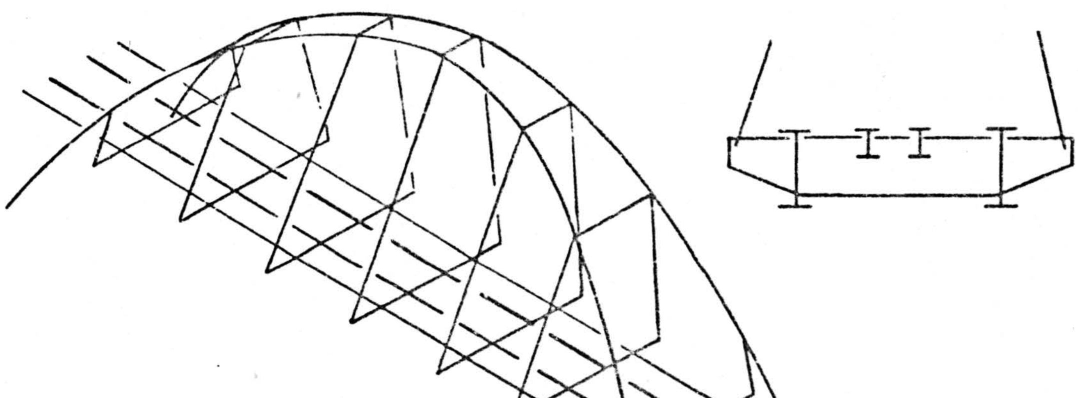


FIGURE 3. ISOMETRIC SCHEMATIC DRAWING OF LEANING ARCHES BRIDGE CONCEPT IN NEW BRIDGE APPLICATION

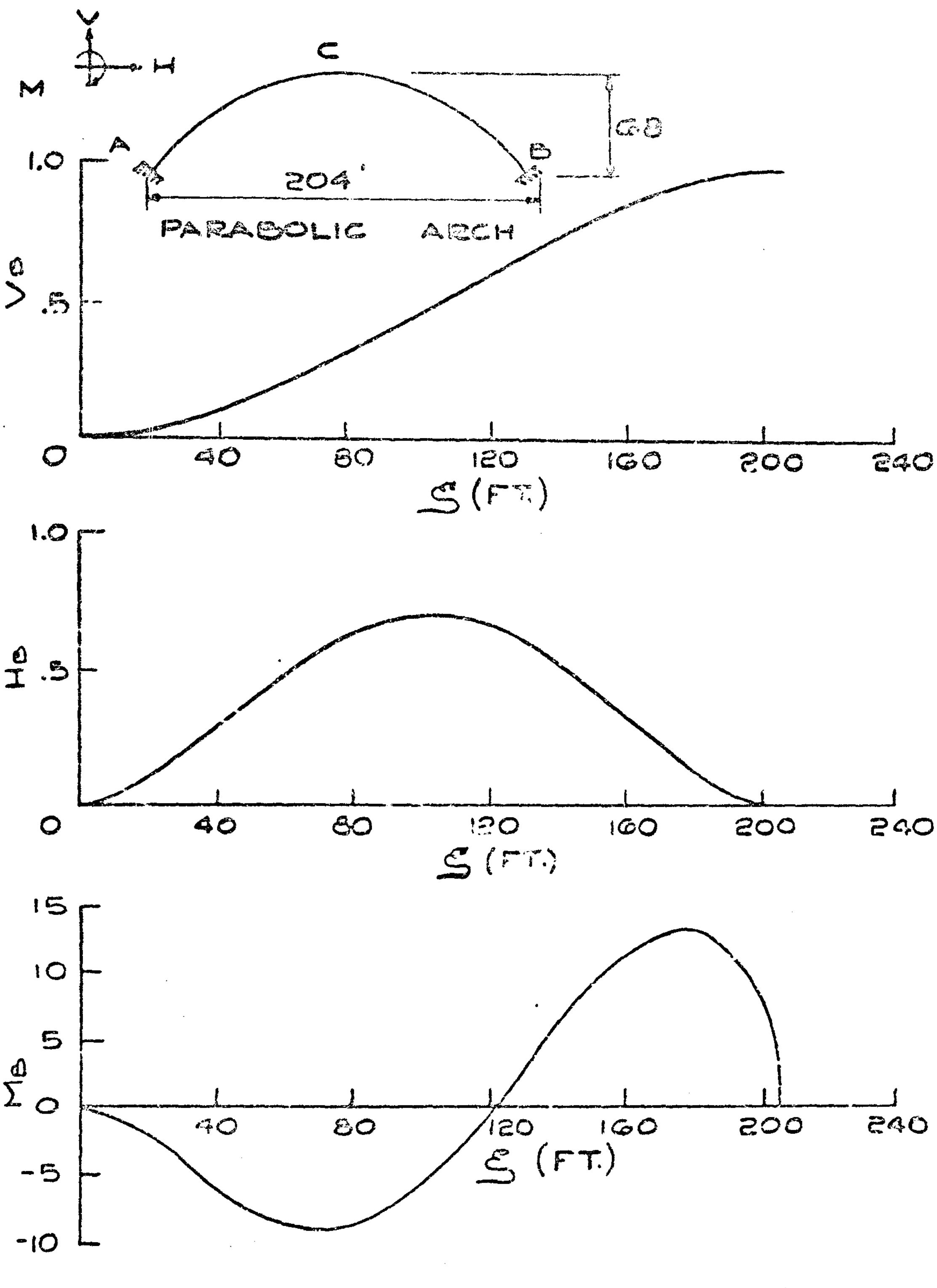


FIGURE 4. INFLUENCE DIAGRAMS FOR LEANING ARCHES BRIDGE

TABLE I. DESIGN VALUES: LEANING ARCHES BRIDGE CONCEPT IN NEW BRIDGE APPLICATION

(a) Principal Structural Members

Members	Concentrated Loads: Equation for Concentrated Live Load Effect	Concentrated Live Load Effect	Impact	Concentrated Live Load Plus Impact	Uniform Loads: Equation for Uniform Load Effect	Dead Load Force Effect	Live Load Force Effect	Live Load Effect Plus Impact Effect	Total Effect for Design
Arches	Moments								
					$M_{DL} = 57.5w^{(4)}$	172.5 K-ft			919.5 K-ft
	$M_{max} = 7.15P^{(1)}$	260 K-ft	$1.33^{(3)}$	346 K-ft	$M_{LL} = 236w^{(4)}$		30[illegible] K-ft	401 K-ft	
	Thrusts:								
	$F_{max} = 0.58P^{(1)}$	20.7 K	$1.33^{(3)}$	27.5 K	$F = 71.0w^{(4)}$	213 K	91 K	121 K	361.5 K
Tension Members									
T_{1-7}	Axial Force: $F = 0.61P^{(1)}$	21.9 K	1.33	29.1 K	$F = 15.7w^{(4)}$	47.0 K	20.1 K	26.7 K	102.6 K

(1) P = 36 K as concentrated load from two lanes.
(2) Arch moment and thrust equations obtained from method in Borg and Gennaro, Advanced Structural Analysis, D. Van Nostrand Co Inc, 1959.
(3) Span for impact computation = 25.5 ft.
(4) w_{DL} = 3000 plf and w_{LL} = 1280 plf as uniform loads for two lanes.

(b) Floor System Members

Member	Concentrated Loads: Equation for Concentrated Live Load Effect	Concentrated Live Load Effect	Impact	Concentrated Live Load Plus Impact	Uniform Loads: Equation for Uniform Load Effect	Dead Load Force Effect	Live Load Force Effect	Live Load Effect Plus Impact Effect	Total Effect for Design
FB_{1-7}	Moment: $M = 10.8P^{(1)}$	389 K-ft	$1.33^{(2)}$	517 K-ft	$M = 138w^{(3)}$	414 K-ft	177 K-ft	235 K-ft	1166 K-ft
	Shear: $V = 0.50P^{(1)}$	18 K	$1.33^{(2)}$	23.9 K	$V = 12.9w^{(3)}$	38.7 K	16.5 K	21.9 K	84.5 K
Stringer (typical of 16)	Moment: $M = 5.2P^{(4)}$	63.7 K-ft	1.33	84.6 K-ft	$M_{DL} = 50.1w^{(5)}$	37.6 K-ft	28.0 K-ft	37.2 K-ft	159.4 K-ft
	Shear: $V = 1.00P^{(4)}$	9.0 K	1.33	12.0 K	$M_{LL} = 64.0w^{(5)}$	9.7	5.6 K	7.5 K	29.2 K
					$V = 12.9w^{(5)}$				

(1) P is concentrated load for two lanes, load is considered to act at center of floor beam, P = 36 K.
(2) Span = 25.5 ft for floor beam impact computation.
(3) w_{DL} = 3000 plf and w_{LL} = 1280 plf as est Dead Load and Live Load for two lanes, acting at center of floor beam.
(4) P is equivalent wheel load on single stringer, assumed to be 12.3 K for moment, 17.7 K for shear.
(5) w = 750 plf est Dead Load and 935 plf Live Load.

TABLE II. PRELIMINARY DESIGN: LEANING ARCHES BRIDGE CONCEPT IN NEW BRIDGE APPLICATION

Member	Design Value	Design Notes	Section	Area	Unit Weight	Length	Weight	Quantity	Total Weight
Principal Structural Members									
Arches									
Moments	M_{max} = 919.5 K-ft F_{max} = 361.5 K	Design as two box girders leaning inward and joining at crown.	Box girder 42 × 21-in. 21 × 1-in. flange 5/16-in. webs.	67 sq in.	226 plf	250 ft	56.5 K	2	113 K
Tension Members									
T_1: T_2: T_3: T_4: T_5: T_6: T_7:	102.8 K	Design as tension member for 102.8 K load, allowable stress 20 ksi, A req'd = 0.514 sq in.	Use 2-3/4-in. dia cable	--	17.5 plf	9 ft 30 ft 42 ft 47 ft 42 ft 30 ft 9 ft	0.18 K 0.53 K 0.74 K 0.82 K 0.74 K 0.53 K 0.18 K	2 2 2 2 2 2 2	0.4 K 1.0 K 1.5 K 1.6 K 1.5 K 1.0 K 0.4 K
FB: Moment Shear	1166 K-ft 84.5 K-ft	Plate girder design indicated, S req'd = 702 cu in.	41 × 14 in., 14 × 1/14-in. flanges, 5/16-in. webs.	47.50 sq in.	162 plf	54 ft	8.8 K	7	61.5 K
Stringers: Moment Shear	159.4 K-ft 29.2 K	Use WF beam, S req'd = 206 cu in.	21 WF 55	--	55 plf	204 ft	11.2 K	4	45 K 227.0
						Bracing, diaphragms (15%)			34
						Total Bridge Weight			261.0 K

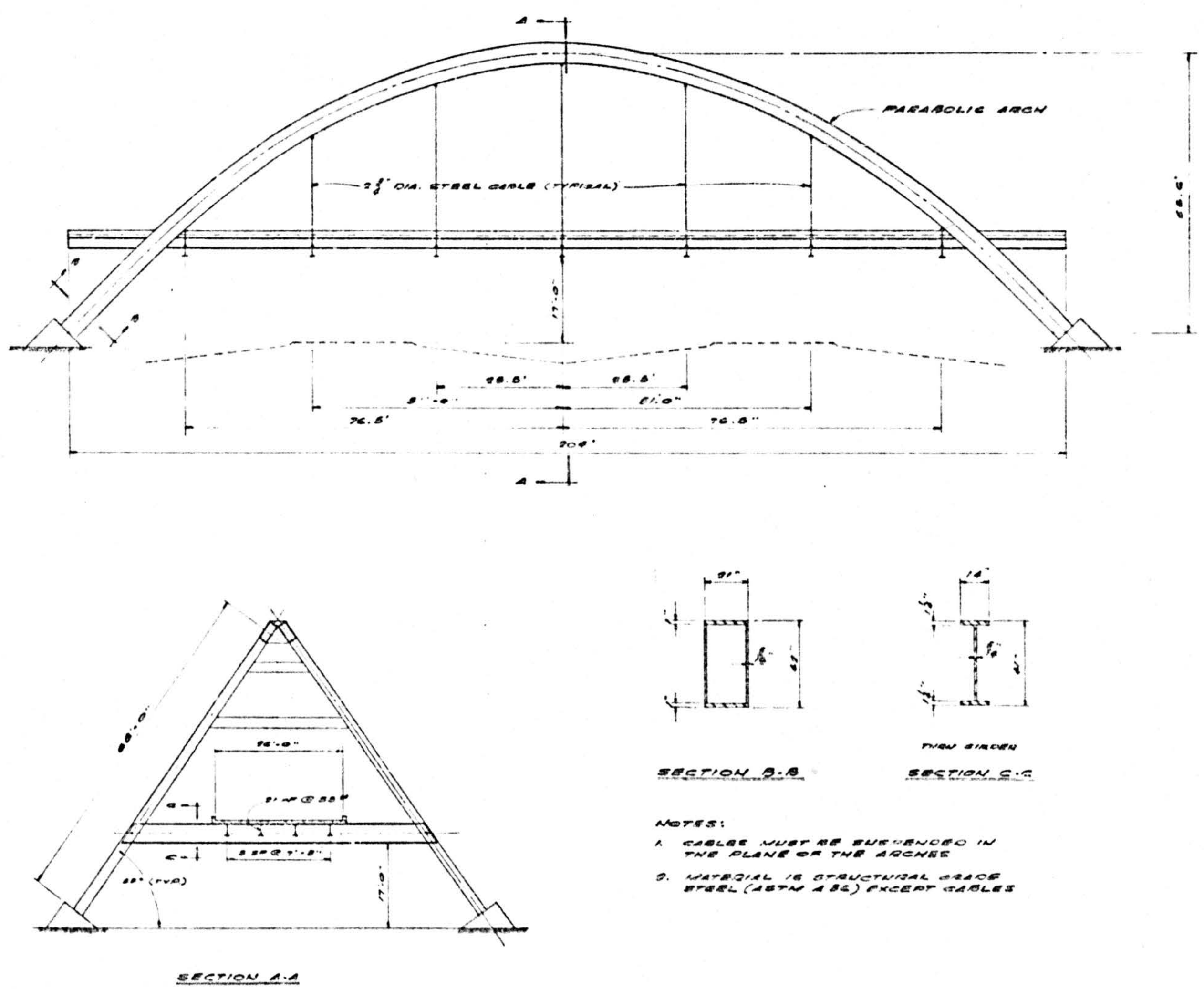


FIGURE 5. PRELIMINARY DESIGN DRAWING OF LEANING ARCHES BRIDGE CONCEPT IN NEW BRIDGE APPLICATION

TABLE III. PRELIMINARY COST ESTIMATES: LEANING ARCHES BRIDGE CONCEPT IN NEW BRIDGE APPLICATION

Item	Unit	Quantity(1)	Unit Cost(2)	Cost: Prototype Only	Cost: Quantity
1. Concrete (slab)	cy	123.5	$71.00	$ 8,770	
2. Reinforcing Steel	lb	29,600	0.12	3,550	
3. Structural Steel, I Beam	lb	45,000	0.20	9,000	
4. Structural Steel, Girder	lb	61,500	0.25	15,400	
5. Structural Steel, Fab. (Arch)	lb	113,000	0.50(0.30)(3)	56,500	($ 33,900)
6. Steel Cable	lb	7,500	0.75(0.65)(3)	5,630	(4,870)
7. Misc. Steel	lb	34,000	0.30	10,200	
8. Bridge Rail	lf	404	15.00	6,050	
Total Estimated Cost (Superstructure Only)				$111,000	($ 91,740)

(1) Items 1 and 2 from Texas Highway Department Concrete Deck Standards, continuous I beam unit, 0° skew, H20-S16 loading; Items 3 through 7 from Table II.
(2) Items 1 through 4, 7, and 8 from first quarter (CY 69) Texas Highway Department construction cost data for in place units shown. These components are conventional bridge construction items.
(3) Items 5 and 6 are estimated in place unit costs for one-of-a-kind bridge (prototype) installations. Procurement in quantity costs (unit and total) are shown in parentheses.

B. Bridle Bridge Concept

The Bridle Bridge concept is an unsymmetrical bridge patterned after a bridge over the River Tartaro, Canada. This concept employs cables to provide an intermediate support for a conventional plate girder bridge deck installation, as shown in the artist's sketch in Figure 6. Cable forces developed in providing support of the plate girders are reacted by a vertical rigid frame and another set of cables terminating at anchorages off of the structure. The plate girder may be continuous or contain a simple span. Both of these structural schemes are considered in the discussions of analysis, preliminary design, and cost estimates in the following paragraphs.

1. Application Discussion

The Bridle Bridge is an unsymmetrical structure which requires anchorages approximately 40 to 50 feet beyond the end of the bridge. This may restrict use of this bridge concept at certain sites, while the unsymmetrical configuration may be advantageous at others. A site where this bridge concept could be effectively employed is suggested by intricate, multi-level interchanges. The section of the Bridle Bridge which does not require vertical supports, above or below the roadway, could be suspended under a crossing structure and/or over a crossed structure to minimize support interference requirements.

Applications of Bridle Bridge girder configuration which contain a hinge are essentially the same as for the continuous girder configuration. Site conditions may exist where the reduced girder depth, that may be realized in the area of the simple span, would be of sufficient benefit to warrant employing the "hinged" concept. In designing the girder with a hinge for a given length of span, the designer has two key points in the structure which can be varied to optimize the structural configuration for specific requirements: (1) the hinge location, and (2) the location of the cable support on the plate girder.

2. Analysis Discussion

This structure pictured schematically in Figure 7 is statically indeterminate in the continuous girder configuration; thus, a computer oriented solution for determining design values is desirable. A method of analysis for the continuous plate girder configuration is contained in Appendix B. A computer program written to assist in analyzing the Bridle Bridge structure in the continuous plate girder configuration is contained in Appendix E, along with output information for the specific bridge considered in the preliminary design. The output from this program provided influence line data points which permit easy determination of the critical section for each principal structural member. A more generalized computer program, with AASHO Specification loading and a variable geometry capability, would provide the designer with a complete load/deflection spectrum for a variety of roadway geometric requirements for use in final design studies.

The Bridle Bridge structure configured with a hinge in the plate girder is statically determinate. A method of analysis for this plate girder configuration is presented in Appendix B. The specific geometry selected (i.e., the specific locations of the hinge and cable support) is not the result of a detailed parametric study, although examination of the influence lines indicates that the design would not be changed substantially as a result of design optimization efforts.

Influence lines for the Bridle Bridge, continuous girder configuration and hinged girder configuration, are presented in Figures 8 and 9, respectively. Using these influence lines, design values were determined using the AASHO loading; these values are summarized in Tables IV and V. The vertical tower experiences no bending forces due to the vertical loads employed in this preliminary design, although lateral loads due to wind and other unsymmetrical loading conditions could produce significant bending loads.

3. Preliminary Design Discussion

Using the design values in Tables IV and V, preliminary designs were developed for the continuous and hinged girder Bridle Bridge configurations; these preliminary designs are summarized in Tables VI and VII. The primary structure consists of two pairs of cables, a vertical tower, and horizontal plate girders. Intermediate transverse floor beams support the interior stringers and frame into the plate girders. The vertical tower is constructed of steel plate with a box cross section, although other materials such as concrete and aluminum could also be employed. The plate girders, floor beams, and stringers are of conventional steel construction. The thrust forces developed in the plate girders due to the cable loads are significant, and consideration of these forces in designing the abutments is required. The designer has the option of restraining the structure at the abutment nearest the tower (producing compression in the girders due to the horizontal force component in the cable) or fixing the structure at the opposite abutment (producing tension in the girder due to the horizontal force component in the cable). A substructure analysis would be required to develop the most effective scheme for a particular site.

A design sketch illustrating the preliminary design of the continuous girder Bridle Bridge is shown in Figure 10. For the hinged girder configuration, the hinge and cable tie locations are as shown in the preliminary design for this configuration in Figure 11. In this preliminary design, the bending moment distribution was such that the section depth required for the simple span is less than that required for the "balanced element." The two variables (hinge and cable tie) could be moved to yield sufficiently equal maximum design moments to permit utilization of a constant depth section; however, this situation may not represent the "optimum" condition from a weight standpoint.

4. Cost Discussion

Preliminary cost estimates for two configurations of a prototype Bridle Bridge (superstructure only) were prepared and are summarized in Table VIII. Estimates were prepared for both the continuous and hinged Bridle Bridge configurations, using materials quantities from Tables VI and VII. Unit prices were taken from current* Texas Highway Department bid averages, with the exception of unit prices used for computing costs of the tower and cables. These prices were adjusted to reflect consideration of the unconventional fabrication and erection procedures required for the prototype structure. Unit prices were also developed which reflect conventional construction prices for procurement in quantity, subsequent to construction of a prototype. These unit prices are shown in parentheses. It is possible that the vertical tower for a prototype structure will not require as large an adjustment as shown for prototype construction, but a factor identical to that employed in the Leaning Arches Bridge preliminary cost estimate was used to remain consistent with other preliminary cost estimates.

*First quarter, calendar year 1969.

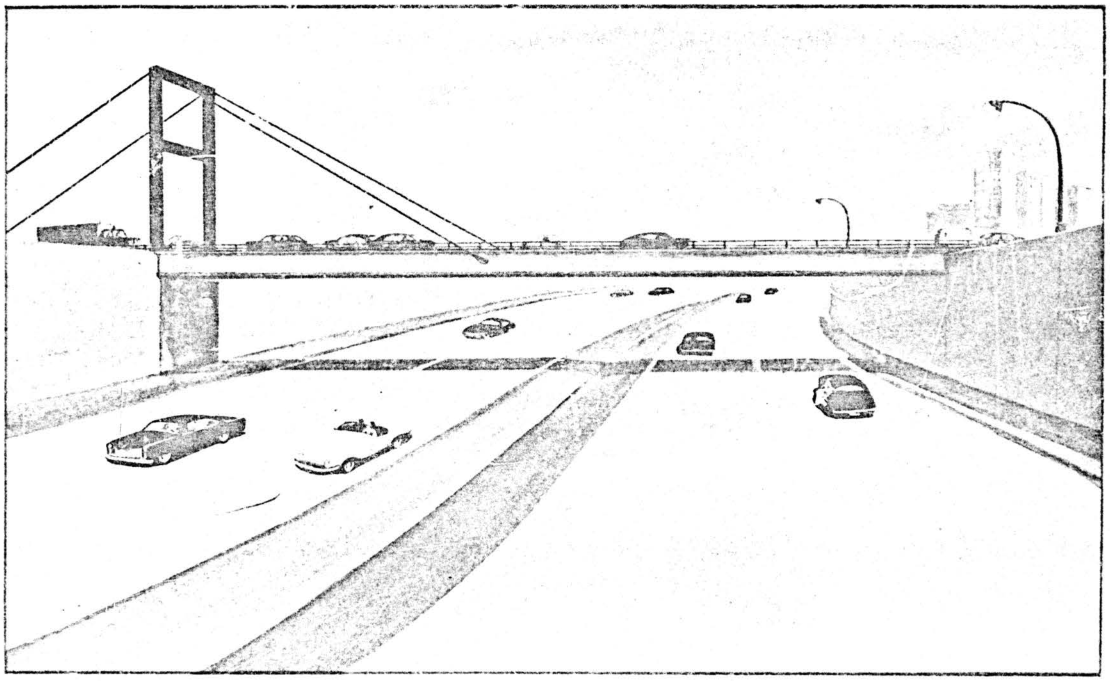


FIGURE 6. ARTIST'S SKETCH OF BRIDLE BRIDGE

FIGURE 7. ISOMETRIC SCHEMATIC DRAWING OF BRIDLE BRIDGE CONCEPT IN NEW BRIDGE APPLICATION

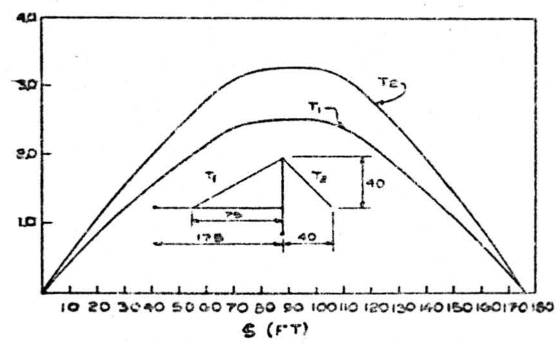


FIGURE 8. INFLUENCE DIAGRAMS FOR BRIDLE BRIDGE
(CONTINUOUS GIRDER CONFIGURATION)

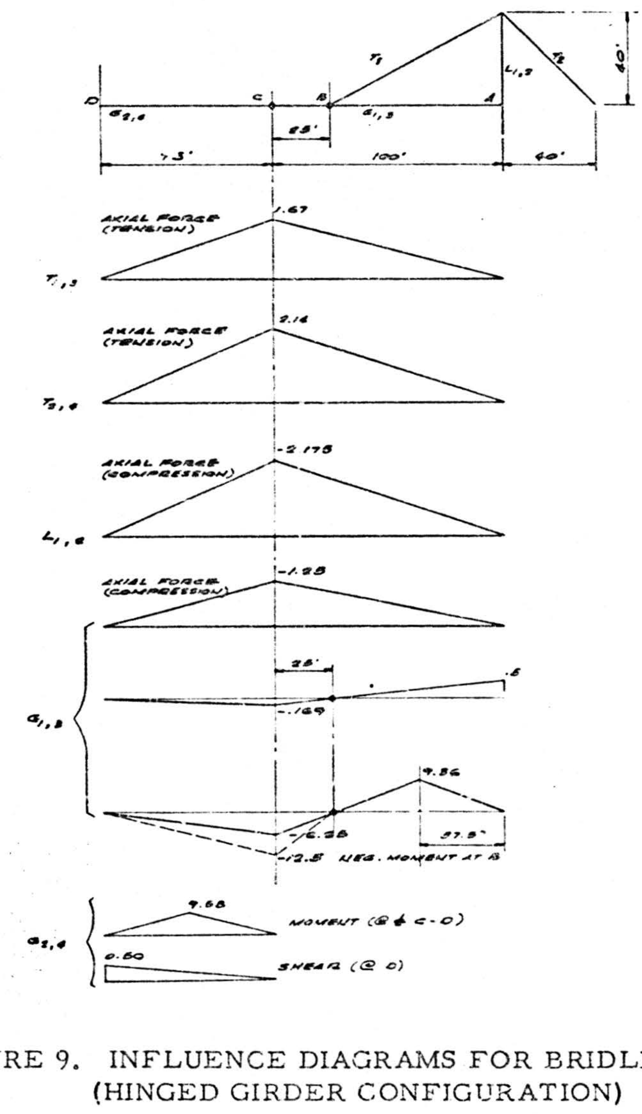


FIGURE 9. INFLUENCE DIAGRAMS FOR BRIDLE BRIDGE
(HINGED GIRDER CONFIGURATION)

TABLE IV. DESIGN VALUES: BRIDLE BRIDGE CONCEPT IN NEW BRIDGE APPLICATION (CONTINUOUS GIRDER CONFIGURATION)

(a) Principal Structural Members

	Concentrated Loads				Uniform Loads				
Member	Equation for Concentrated Live Load Effect	Concentrated Live Load Effect	Impact	Concentrated Live Load Plus Impact	Equation for Uniform Load Effect	Dead Load Force Effect	Live Load Force Effect	Live Load Effect Plus Impact Effect	Total Effect for Design
T_1	Axial Force: $F = 1.25P^{(1)}$	46.2 K	$1.17^{(2)}$	54.0 K	$F = 146.0w$	497 K	187 K	219 K	770.0 K
T_2	Axial Force: $F = 1.54P^{(1)}$	55.5 K	$1.17^{(2)}$	65.0 K	$F = 180.0w$	612 K	230 K	269 K	946.0 K
$L_{1,2}$	Axial Force: $F = -1.59P^{(1)}$	-57.2 K	$1.17^{(2)}$	-66.9 K	$F = -184.8w$	-627 K	-237 K	-277 K	-970.9 K
$G_{1,3}$	Axial Force: $F = -1.15P^{(1)}$	41.4 K	1.1	46.4 K	$F = -134.0w$	-455 K	-171 K	-200 K	-703.4 K
	Moment (max. positive): $F = 9.85P$	356 K-ft	1.16	391 K-ft	$M = 400w$	1360 K-ft	511 K-ft	598 K-ft	2372.0 K-ft
	Moment (max. negative): $F = -4.9P$	-450 K-ft	1.16	-522 K-ft	$M = -500w$	-1700 K-ft	-640 K-ft	-750 K-ft	-2656 K-ft
	Shear: $F = -0.35$	12.6 K	1.17	14.7 K	$V = 60.3w$	206 K	77.2 K	90.3 K	311.0 K

(1) P is concentrated load at transverse center of bridge representing two lane loads. F = 36 K.
(2) Span = 175 ft for impact computation.
(3) w = 3400 plf (DL est) and 1280 plf (LL) representing two lanes.

(b) Floor System Members

	Concentrated Loads				Uniform Loads				
Member	Equation for Concentrated Live Load Effect	Concentrated Live Load Effect	Impact	Concentrated Live Load Plus Impact	Equation for Uniform Load Effect	Dead Load Force Effect	Live Load Force Effect	Live Load Effect Plus Impact Effect	Total Effect for Design
Floor beam (typical of 6)	Moment: $F = 7.50P^{(1)}$	270 K-ft	$1.29^{(2)}$	349 K-ft	$F = \frac{1}{2}(50)(7.50)w^{(3)}$	527 K-ft	241 K-ft	311 K-ft	1187 K-ft
	Shear: $F = 0.50P^{(1)}$	18.0 K	$1.29^{(2)}$	23.2 K	$F = \frac{1}{2}(50)(0.50)w^{(3)}$	35.0 K	16.0 K	20.6 K	78.8 K-ft
Stringers (typical of 21)	Moment: $F = 6.25P^{(4)}$	76.8 K-ft	1.3	100.0 K-ft	$F = \frac{1}{2}(25)(6.25)w^{(5)}$	43.6 K-ft	34.4 K-ft	44.7 K	188.3 K-ft
	Shear: $F = 1.00P^{(4)}$	17.7 K	1.3	23.0 K	$F = \frac{1}{2}(25)(1.00)w^{(5)}$	7.1 K	5.5 K	7.2 K	35.7 K

(1) P is concentrated load for two lanes acting at center of floor beam; P = 36 K.
(2) Span is 50 ft for impact factor computation.
(3) w_{DL} = 2800 (est) and w_{LL} = 1280 plf as Dead Load and Live Load for two lanes.
(4) P is equivalent wheel load on single stringer, assumed to be 12.3 K for moment, 17.7 K for shear.
(5) w_{DL} = 560 plf (est), w_{LL} = 440 plf AASHO Distribution.

TABLE V. DESIGN VALUES: BRIDLE BRIDGE CONCEPT IN NEW BRIDGE APPLICATION (HINGED GIRDER CONFIGURATION)

(a) Principal Structural Members

	Concentrated Loads				Uniform Loads				
Member	Equation for Concentrated Live Load Effect	Concentrated Live Load Effect	Impact	Concentrated Live Load Plus Impact	Equation for Uniform Load Effect	Dead Load Force Effect	Live Load Force Effect	Live Load Effect Plus Impact Effect	Total Effect for Design
T_1	Axial Force: $F = 1.67P^{(1)}$	60.0 K	$1.16^{(2)}$	69.6 K	$F = \frac{1}{2}(1.67)(175)w$	497 K	187 K	217 K	753.6 K
T_2	Axial Force: $F = 2.14P^{(1)}$	77.0 K	$1.16^{(2)}$	89.3 K	$F = \frac{1}{2}(2.14)(175)w$	635 K	239 K	278 K	1002.3 K
$L_{1,2}$	Axial Force: $F = 2.175P^{(1)}$	-78.3 K	$1.16^{(2)}$	-91.0 K	$F = \frac{1}{2}(2.175)(175)w$	-647 K	-244 K	-283 K	-1021 K
$G_{1,3}$	Axial Force: $F = -1.25P^{(1)}$	-45.0 K	1.16	-52.2 K	$F = \frac{1}{2}(-1.25)(175)w$	-370 K	-140 K	-163 K	-613.0 K
	Moment (max. positive): $F = 9.38P$	337 K-ft	1.16	391 K-ft	$M = \frac{1}{2}(9.38)(75)w_{LL}$ $= (39)w_{DL}$	132.5 K-ft	450 K-ft	524 K-ft	1047.5 K-ft
	Moment (max. negative): $F = -12.50P$	-450 K-ft	1.16	-522 K-ft	$M = \frac{1}{2}(-12.50)(100)w$	-2120 K-ft	-800 K-ft	-929 K-ft	-3166 K-ft
	Shear: $F = 0.55P$	18 K	1.16	21.1 K	$V = \frac{1}{2}(0.50)(75)w_{LL}$ $V = (10.55)w_{DL}$	36.0 K	24.0 K	28.8 K	85.9 K
$G_{2,4}$	Moment: $F = 9.38P$	337 K-ft	1.16	391 K-ft	$M = \frac{1}{2}(9.38)(75)w$	1190 K-ft	450 K-ft	524 K-ft	2105 K-ft
	Shear: $F = 0.50P$	18 K	1.16	20.9 K	$V = \frac{1}{2}(0.5)(75)w$	63.8 K	24.0 K	28.8 K	113.5 K

(1) P is concentrated load at transverse center of bridge representing two lane loads; P = 36 K.
(2) Span = 184 ft for impact computation.
(3) w = 3400 plf (DL est) and 1280 plf (LL) representing two lanes.

(b) Floor System Members

	Concentrated Loads				Uniform Loads				
Member	Equation for Concentrated Live Load Effect	Concentrated Live Load Effect	Impact	Concentrated Live Load Plus Impact	Equation for Uniform Load Effect	Dead Load Force Effect	Live Load Force Effect	Live Load Effect Plus Impact Effect	Total Effect for Design
Floor beam (typical of 6)	Moment: $M = 7.50P^{(1)}$	270 K-ft	$1.29^{(2)}$	349 K-ft	$F = \frac{1}{2}(50)(7.50)w^{(3)}$	527 K-ft	241 K-ft	311 K-ft	1187 K-ft
	Shear: $F = 0.50P^{(1)}$	18.0 K	$1.29^{(2)}$	23.2 K	$F = \frac{1}{2}(50)(0.50)w^{(3)}$	35.0 K	16.0 K	20.6 K	78.8 K-ft
Stringers (typical of 21)	Moment: $M = 6.25P^{(4)}$	76.8 K-ft	1.3	100.0 K-ft	$F = \frac{1}{2}(25)(6.25)w^{(5)}$	43.6 K-ft	34.4 K-ft	44.7 K	188.3 K-ft
	Shear: $F = 1.00P^{(4)}$	17.7 K	1.3	23.0 K	$F = \frac{1}{2}(25)(1.00)w^{(5)}$	7.1 K	5.5 K	7.2 K	35.7 K

(1) P is concentrated load for two lanes acting at center of floor beam. P = 36 K.
(2) Span is 50 ft for impact factor computation.
(3) w_{DL} = 2800 (est) and w_{LL} = 1280 plf as Dead Load and Live Load for two lanes.
(4) P is equivalent wheel load on single stringer, assumed to be 12.3 K for moment, 17.7 K for shear.
(5) w_{DL} = 560 plf (est), w_{LL} = 440 plf (AASHO Distribution).

TABLE VI. PRELIMINARY DESIGN: BRIDLE BRIDGE CONCEPT IN NEW BRIDGE APPLICATION (CONTINUOUS GIRDER CONFIGURATION)

Member	Design Value	Design Notes	Section	Area	Unit Weight	Length	Weight	Quantity	Total Weights
Principal Structural Members									
T_1: Axial Force	770.0 K	Tensile member, use cable with 80-ksi allowable; A req'd = 9.62	3-1/2-in. dia.	9.62 sq in.	32.7 plf	85.0 ft	2.8 K	2	5.6 K
T_2: Axial Force	946.0 K	Tensile member, use cable with 80-ksi allowable; A req'd = 11.8	3-7/8-in. dia.	11.79 sq in.	40.0 plf	56.5 ft	2.3 K	2	4.6 K
$L_{1,2}$:	-970.9 K	Compression member, design as column. L = 57 ft, radius of gyration approximately 10 in., allowable stress = 14 ksi	24 × 24 × 3/4-in. square box column	70 sq in.	238 plf	57 ft	13.6 K	2	27.2 K
$G_{1,3}$: Axial Force Moment Shear	 -70[illegible].4 K -2656 K-ft [illegible]11 K	Negative moment at T_2 attach point governs design; design a uniform section plate girder, S req'd = 1590 cu in.; axial load stress approximately 5500 psi, therefore, increase S req'd approximately 25%	72 × 24-in., 24 × 1-1/2-in. flange, 3/8 in. web	106.5 sq in.	362 plf	175 ft	63.1 K	2	126.2 K
						Principal structure			163.6 K
						Appurtenances, etc.			32.7 K
						Total principal structure			196.3 K
Floor System Members									
FB: Moment Shear	 1187 K-ft 53.6 K	Short, deep beam, S req'd = 713 cu in.; use WF beam	36 WF 230	--	230 plf	30 ft	6.9 K	7	48.3 K
Stringers: Moment Shear	 168.3 K-ft 35.7 K	Use WF beam, S req'd = 113 cu in.	21 WF 62	--	62 plf	25 ft	1.6 K	21	33.6 K
						Floor system			81.9 K
						Appurtenances, diaphragms (15%)			12.3 K
						Total floor system			94.2 K
						Total Bridge Weight			290.5 K

TABLE VII. PRELIMINARY DESIGN: BRIDLE BRIDGE CONCEPT IN NEW BRIDGE APPLICATION (HINGED GIRDER CONFIGURATION)

Member	Design Value	Design Notes	Section	Area	Unit Weight	Length	Weight	Quantity	Total Weights
Princip. Structural Members									
T_1: Axial Force	753.6 K	Tensile member, use cable with 80-ksi allowable; A req'd = 9.43	3-1/2-in.-dia. cable	9.621 sq in.	32.7 plf	85.0 ft	2.8 K	2	5.6 K
T_2: Axial Force	1002.5	Tensile member, use cable with 80-ksi allowable; A req'd = 12.5	4-in.-dia. cable	12.57 sq in.	42.7 plf	56.5 ft	2.4 K	2	4.8 K
$L_{1,2}$	-1021.0 K	Compression member, design as column: L = 57 ft, radius of gyration approximately 10 in., allowable stress = 14 ksi	24 × 24 × 3/4-in. square box column	70 sq in.	238 plf	57 ft	13.6 K	2	27.2 K
$G_{1,3}$: Axial Force Moment Shear	 -613.0 K -3166 K-ft 85.9 K	Negative moment at T_2 attach point governs design; design a uniform section plate girder, S req'd = 1900 cu in.; axial load stress approximately 5500 psi, therefore, increase S req'd approximately 40%	72 × 26-in., 26 × 1-1/2-in. flange, 3/8-in. web, I = 97,000	112.5 sq in.	382 plf	100 ft	38.2 K	2	76.4 K
$G_{2,4}$: Moment Shear	 2105 K-ft 113.5 K	Positive moment at mid-span; design as uniform section plate girder, S req'd = 1260	64 × 20-in., 20 × 1-in. flange 3/8-in. web	63.2 sq in.	214 plf	75 ft	16.0 K	2	32.0 K
						Principal structure			146.0 K
						Appurtenances, etc. (20%)			28.2 K
						Total principal structure			174.2 K
Floor System Members									
FB: Moment Shear	 1167 K-ft 78.8 K	Short, deep beam, S req'd 713 cu in.; use WF beam	36 WF 230	--	230 plf	30 ft	6. K	7	48.3 K
Stringers: Moment Shear	 166.3 K-ft 35.7 K	Use WF beam, S req'd 113 cu in.	21 WF 62	--	62 plf	25 ft	1.6 K	21	33.6 K
						Floor system			81.9 K
						Appurtenances, diaphragms (15%)			12.3 K
						Total floor system			94.2 K
						Total Bridge Weight			268.4 K

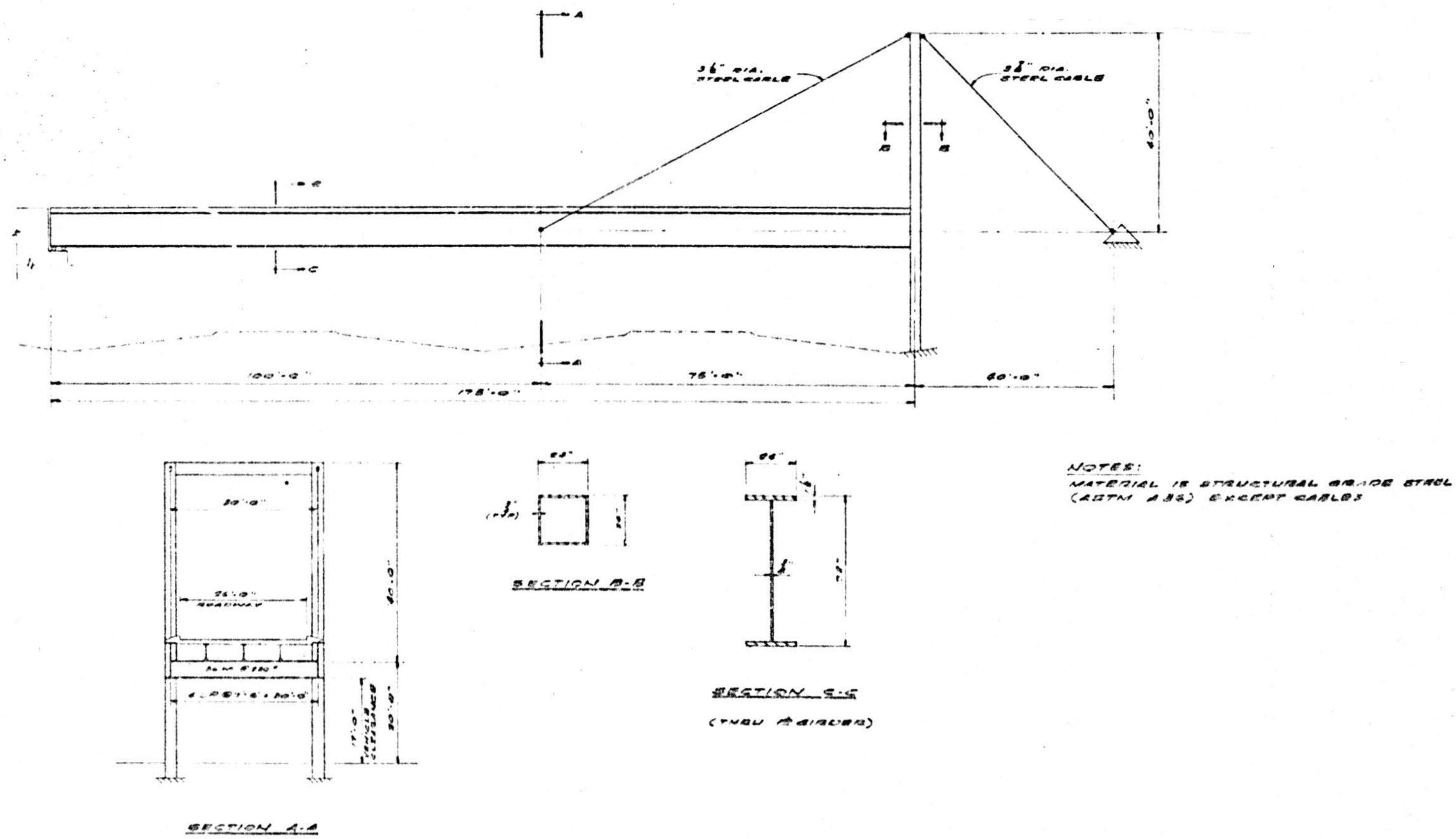


FIGURE 10. PRELIMINARY DESIGN DRAWING OF BRIDLE BRIDGE CONCEPT IN NEW BRIDGE APPLICATION (CONTINUOUS GIRDER CONFIGURATION)

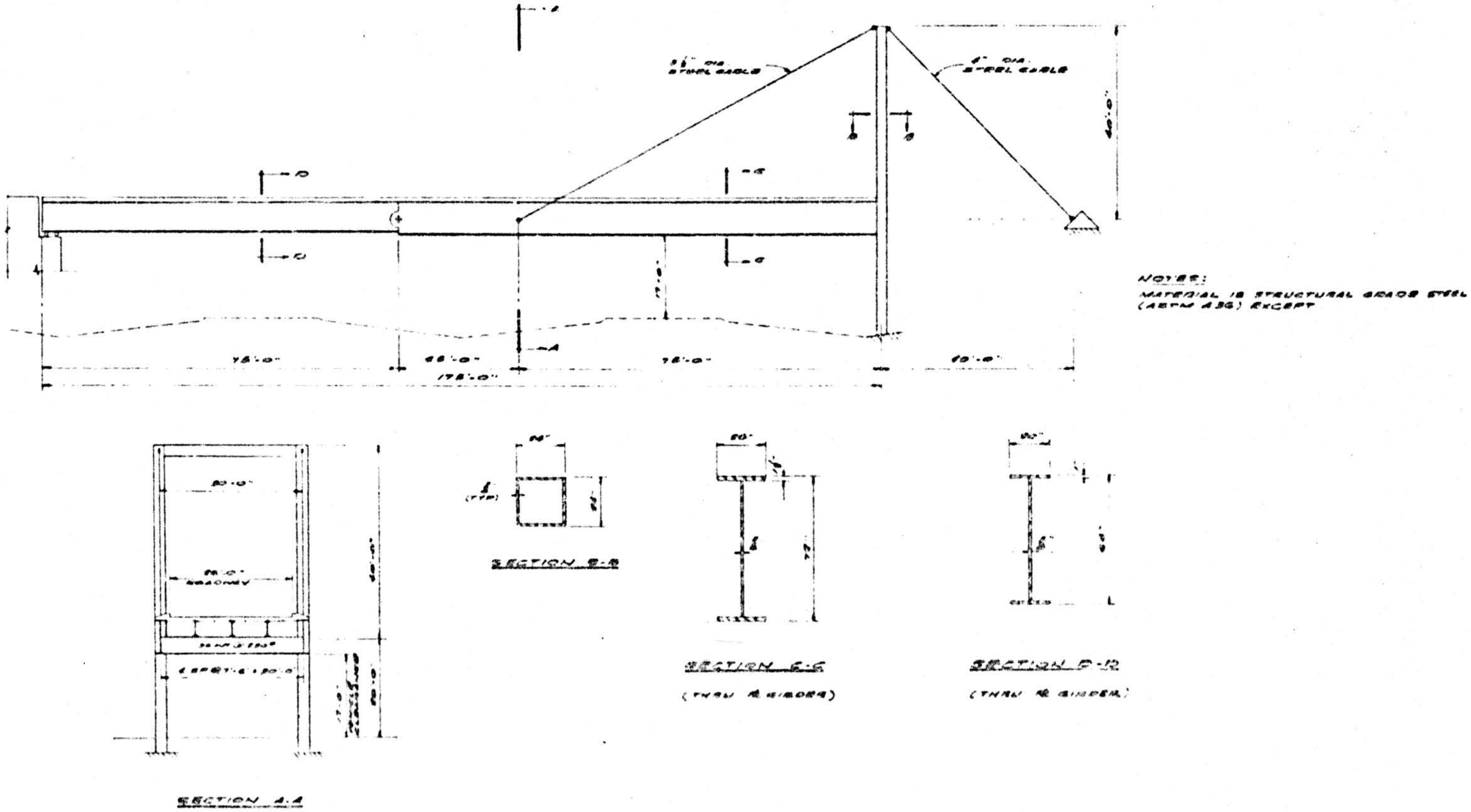


FIGURE 11. PRELIMINARY DESIGN DRAWING OF BRIDLE BRIDGE CONCEPT IN NEW BRIDGE APPLICATION (HINGED GIRDER CONFIGURATION)

TABLE VIII. PRELIMINARY COST ESTIMATES: BRIDLE BRIDGE CONCEPT IN NEW BRIDGE APPLICATION

(a) Continuous Girder Configuration

Item	Unit	Quantity(1)	Unit Cost(2)	Cost: Prototype Only	Cost: Quantity
1. Concrete (slab)	cy	104.0	$71.00	$ 7,370	
2. Reinforcing Steel	lb	24,000	0.12	2,880	
3. Structural Steel, I Beam	lb	81,900	0.20	16,380	
4. Structural Steel, Girder	lb	126,200	0.25	31,500	
5. Structural Steel, Fab. (Tower)	lb	27,200	0.50(0.30)(3)	13,600	($ 8,150)
6. Steel Cable	lb	10,200	0.75(0.65)(3)	7,650	(6,630)
7. Misc. Steel	lb	45,000	0.30	13,500	
8. Bridge Rail	lf	240.0	15.00	5,100	
Total Estimated Cost (Superstructure Only)				$97,980	($91,510)

(b) Girder with Hinge Configuration

Item	Unit	Quantity(1)	Unit Cost(2)	Cost: Prototype Only	Cost: Quantity
1. Concrete (slab)	cy	104.0	$71.00	$ 7,370	
2. Reinforcing Steel	lb	24,000	0.12	2,880	
3. Structural Steel, I Beam	lb	81,900	0.20	16,380	
4. Structural Steel, Girder	lb	108,400	0.25	27,000	
5. Structural Steel, Fab. (Tower)	lb	27,200	0.50(0.30)	13,600	($ 8,150)
6. Steel Cable	lb	10,400	0.75(0.65)	7,800	(6,750)
7. Misc. Steel	lb	40,500	0.30	12,200	
8. Bridge Rail	lf	340.0	15.00	5,100	
Total Estimated Cost (Superstructure Only)				$92,330	($85,[illegible]30)

(1) Items 1 and 2 from Texas Highway Department Concrete Deck Standards, continuous I beam unit, 0° skew, H20-S16 loading; Items 3 through 7 from Tables VI and VII.

(2) Items 1 through 4, 7, and 8 from first quarter (CY 69) Texas Highway Department construction cost data for in place units shown. These components are conventional bridge construction items.

(3) Items 5 and 6 are estimated in place unit costs for one of a kind bridge installations. Procurement in quantity costs (unit and total) are shown in parentheses.

C. Concept Designs for New Bridges

Three additional bridge concepts were identified during the concept review and selection process which may be considered to be potentially effective structural schemes for use in new bridge applications. These three concepts were not subjected to detailed analysis and preliminary design iterations as were the Leaning Arches and Bridle Bridges; nevertheless, they appear to possess certain capabilities for responding to the safety-oriented design criteria. Each of the three concepts may be employed in new bridge construction.* Summaries of engineering data and concept designs emanating from the concept design and evaluation process† involving these three concepts in new bridge applications are included in the following paragraphs. The three concepts are: (1) the Stayed Girder Bridge, (2) the Braced Arch Bridge, and (3) the Leaning Piers Bridge. Included in the concise presentations are sketches, tabulations of key engineering data, concept design sketches, and concept design discussions.

1. Stayed Girder Bridge

The Stayed Girder Bridge, like the Bridle Bridge, is an unsymmetrical, cable-supported structure. It consists of a single continuous girder supported by six cables extending from a single vertical support. The configuration of principal and floor system members considered in developing the concept design is illustrated in Figure 12. This structure is aesthetically pleasing and may be used in siting situations similar to those found advantageous for the Bridle Bridge. A disadvantage in employing this concept concerns the three cable ties which must be anchored off of the structure. These anchorages may interfere with access road locations or right-of-way restrictions.

The structure is statically indeterminate and must be analyzed by considering the complete system, including vertical column, cables, and continuous girders. A method of analysis for this structural scheme which treats the total system is included in Appendix B; this procedure should be employed in developing preliminary designs. The concept design, however, was developed by employing a simplified analysis method based on assumed nonyielding supports and a constant stiffness (EI) girder. The Mueller-Breslau principle[9, 10] was used to obtain the influence diagrams presented in Figure 13 for moments and support reactions in the continuous plate girder.

Support reactions determined from the influence diagrams in Figure 13 were employed to develop design tensile forces in the cables and

*Applications of these concepts to modifications of existing bridges are considered in Section IV (Paragraph C).

†The concept design and evaluation process is described in detail in Volume I.

forces acting on the vertical column. Design values computed for principal structural members are contained in Table IX. The design values determined from the simplified analysis outlined above were employed to develop a concept design. The design procedure employed in developing this concept design did not involve the more detailed iterative process that was used to develop the preliminary designs for the Leaning Arches and Bridle Bridges. Nevertheless, the concept design outlined in Table X and presented in Figure 14 provides an appraisal of the nature of the structure as it would appear if employed in a highway application. Because of the less comprehensive methodology employed in the design of this structure, preliminary cost estimates were not developed.

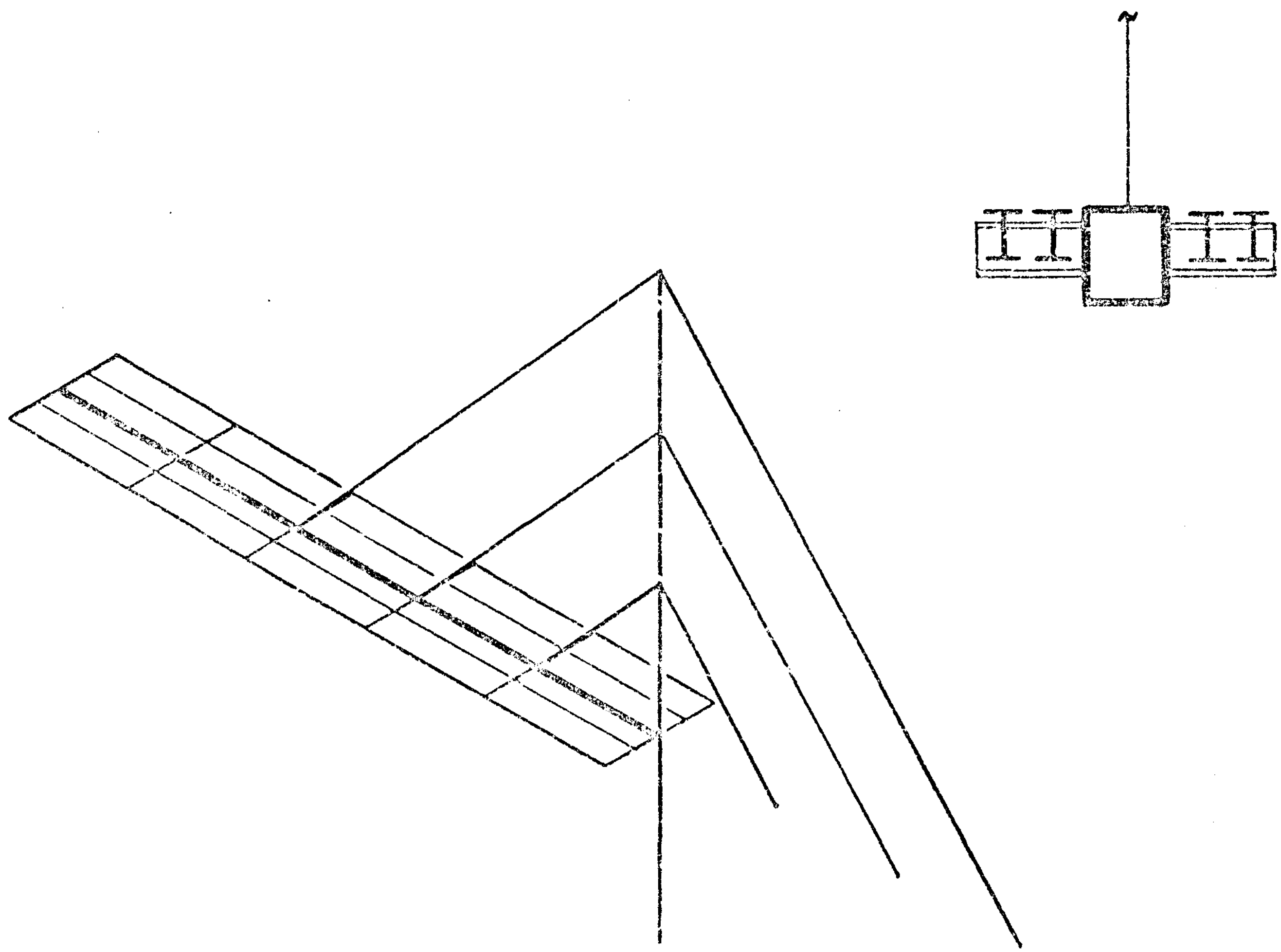


FIGURE 12. ISOMETRIC SCHEMATIC DRAWING OF STAYED GIRDER BRIDGE CONCEPT IN NEW BRIDGE APPLICATION

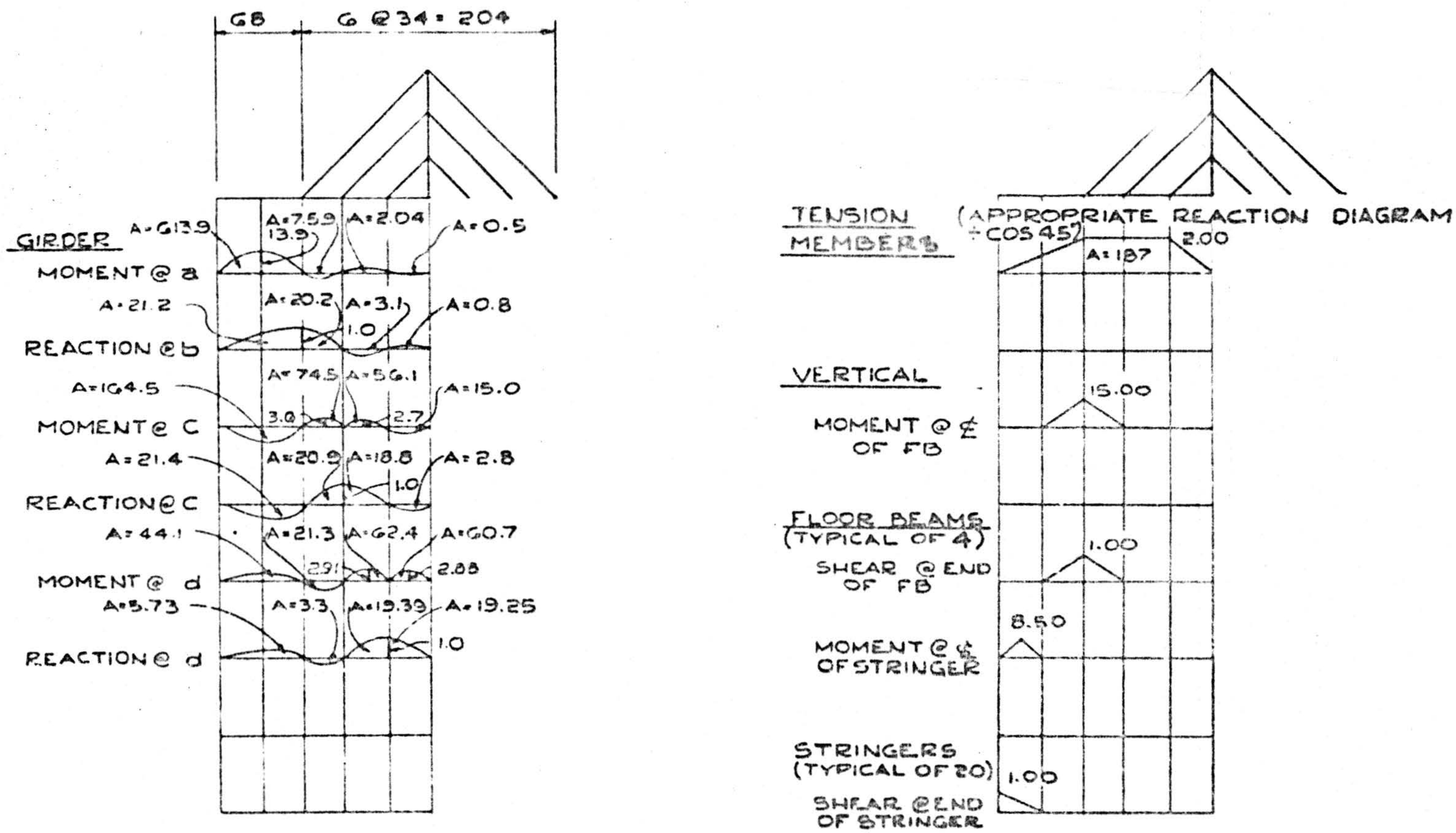


FIGURE 13 INFLUENCE DIAGRAMS FOR STAYED GIRDER BRIDGE

TABLE IX. DESIGN VALUES: STAYED GIRDER BRIDGE CONCEPT IN NEW BRIDGE APPLICATION

(a) Principal Structural Members

Member	Equation for Concentrated Live Load Effect	Concentrated Live Load Effect	Impact	Concentrated Live Load Plus Impact	Equation for Dead Load Effect	Dead Load Effect	Live Load Effect Force	Live Load Effect Plus Impact	Total Effect for Design
	Concentrated Loads				Uniform Loads				
Girder	Moment at g (negative): F = 3.40P[1] (3)	123 K-ft	1.13	139 K-ft	DL: F = Σ × Areas = 121.5w[2] LL: F = Σ × Areas = 138.6w	365 K-ft	177 K-ft	200 K-ft	704 K-ft
	Moment at a (positive): F = 13.9P	501 K-ft	1.13	566 K-ft	DL: F = Σ × Areas = 608.0w LL: F = Σ × Areas = 615.1w	1825 K-ft	769 K-ft	890 K-ft	3281 K-ft
	Moment at f (negative): F = 6.38P[4]	229 K-ft	1.13	259 K-ft	F = 145.0w[2]	435 K-ft	166 K-ft	210 K-ft	904 K-ft
	Axial Force: F = T max cos 45° (est)[5]		--	--	--	--	--	--	149.0 K
Column	Axial Force: F = 2.00P	72 K	1.13	81.1 K	F = 374w[2]	1120 K	478 K	541 K	1733 K
T_{1-10}									
$T_{1,6}$	Axial Force: F = 1.00P[1]	36 K	1.13	40.7 K	DL: F = Σ × Areas = 39.0w LL: F = Σ × Areas = 42.3w	117.3 K	54.1 K	61.4 K	219.4 sec 45° = 310 K
T_2	Axial Force: F = 1.00P	36 K	1.13	40.7 K	F = 34w[6]	102 K	52.4 K	59.2 K	201.9 sec 45° = 285 K
T_3	Axial Force: F = 1.00P	36 K	1.13	40.7 K	DL: F = Σ × Areas = 137.4w[2] LL: F = Σ × Areas = 40.5w	112.3 K	51.8 K	58.8 K	211.8 sec 45° = 299 K
T_4	Axial Force: F = 1.00P	36 K	1.13	40.7 K	DL: F = Σ × Areas = 33.1w LL: F = Σ × Areas = 39.7w	99 K	50.9 K	57.5 K	197.2 sec 45° = 278 K
T_5	Axial Force: F = 1.00P	36 K	1.13	40.7 K	DL: F = Σ × Areas = 38.5w LL: F = Σ × Areas = 41.4w	115.5 K	53.0 K	59.9 K	216.1 sec 45° = 306 K

(1) P = 36 K as two lane concentrated load.
(2) w_{DL} = 3000 plf (est), w_{LL} = 1280 plf as uniform loads for two lanes.
(3) Design values are not computed for points b, c, d, e, f because inspection of influence diagrams (Fig. 34) identifies point a as location of maximum negative moment between b and h.
(4) Moments at right end of first span (68-ft span) estimated as fixed end moments, uniform load, reaction developed as span assigned to $T_2 \times w$.
(5) Estimate axial force in girder as maximum tension member force × cos 45°.

(b) Floor System Members

Member	Equation for Concentrated Live Load Effect	Concentrated Live Load Effect	Impact	Concentrated Live Load Plus Impact	Equation for Uniform Loads Effect	Dead Load Effect	Live Load Effect Force	Live Load Effect Plus Impact	Total Effects for Design
	Concentrated Loads				Uniform Loads				
Floorbeam (typical of 7)									
	Moment: F = 15.00P[1]	270 K-ft	1.26[2]	340 K-ft	$F = \frac{1}{2}(68)(15.00)w$[3]	765 K-ft	326 K-ft	411 K-ft	1516 K-ft
	Shear: F = 1.00P[1]	18 K	1.26[2]	22.7 K	$F = \frac{1}{2}(68)(1.00)w$[3]	51.1 K	21.8 K	27.4 K	101.2 K
Stringers	Moment: F = 8.50P[4]	61.2 K-ft	1.3[5]	79.5 K-ft	$F = \frac{1}{2}(34)(8.50)w$[6]	86.8 K-ft	37.0 K-ft	48.1 K-ft	214.5 K-ft
	Shear: F = 1.00P[4]	10.2 K	1.3[5]	13.3 K	$F = \frac{1}{2}(34)(1.00)w$[6]	10.3 K	4.4 K	5.7 K	29.3 K

(1) P is concentrated load for one lane acting at end of floor beam (18 K).
(2) Span is 68 ft for impact factor consideration.
(3) w_{DL} = 1500 plf (est), w_{LL} = 640 plf as single lane. Dead Load and Live Load acting at end of floor beam.
(4) P is equivalent wheel load on single stringer, assumed to be 7.2 K for moment 1[illegible].2 K for shear.
(5) Span is 34 ft for impact factor computation.
(6) w is uniform load assigned to one stringer: w_{DL} = 750 plf (est); w_{LL} = 256 plf.

TABLE X. CONCEPT DESIGN: STAYED GIRDER BRIDGE CONCEPT IN NEW BRIDGE APPLICATION

Member	Design Value	Design Notes	Section	Area	Unit Weight	Length	Weight	Quantity	Total Weights
Principal Structural Members									
Girder: Moments		Preliminary design as two uniform section box girders. S req'd:							
at a (+)	3281 K-ft	Length o-a-b (68-ft end span), 1970 cu in.	30 × 54-in. box, 1-1/4-in. flange, 5/16-in. webs;	106.9 sq in.	363 plf	68 ft	24.7 K	1	24.7 K
at g (-)	704 K-ft								
at f (-)	904 K-ft	Length b-n (6, 34-ft spans), 542 cu in.	30 × 54-in. box, 1/2-in. flange, 5/16-in. webs	63 sq in.	212 plf	102 ft	21.7 K	1	21.7 K
Column: Axial Force (at base)	1733 K (Nominal Bending)	Design as slender column, allowable stress at base = 10 ksi. A req'd at base = 173 sq in.; taper to 50 sq in. at top.	24 × 24-in. box, 1 in. flanges, 1/2-in. webs, taper to 54 × 54 in. box, 1-in. flanges, 5/8-in. webs	70 sq in. to 170 sq in.	407 plf	102 ft	41.5 K	1	41.5 K
Tension Members:									
$T_{1,6}$	310 K	A req'd = 3.9 sq in. if allowable stress = 80,000 psi	2-1/4-in. cable	3.98 sq in.	13.4 plf	144 ft	1.9 K	2	3.8 K
T_2	285 K					96 ft	1.3	2	2.6 K
T_3	299 K					48 ft	0.7	2	1.4 K
T_4	278 K					48 ft	0.7	2	1.4 K
T_5	306 K					96 ft	1.3	2	2.6 K
						Total principal structure			99.7 K
						Miscellaneous, stiffeners (20%)			20.0 K
									119.7 K
Floor System Members									
F.B.: Moment / Shear	1516 K-ft / 101.2 K	Short deep beam, S req'd = 910 cu in., use plate girder	42 × 22-in., 22 × 1-in. flanges, 5/16-in. web	56.5 sq in.	192 plf	30.[illegible] ft	5.9 K	4	23.6 K
Stringers: Moment / Shear	214.5 K-ft / 29.3 K	Design as WF beam, S req'd = 129 cu in.	21 WF 62	--	62 plf	34	2.1 K	20	42.0 K
						Total floor system			65.6 K
						Diaphragms, bracing, etc. (20%)			13.[illegible] K
									76.6 K
						Total Bridge Weight			196.5 K

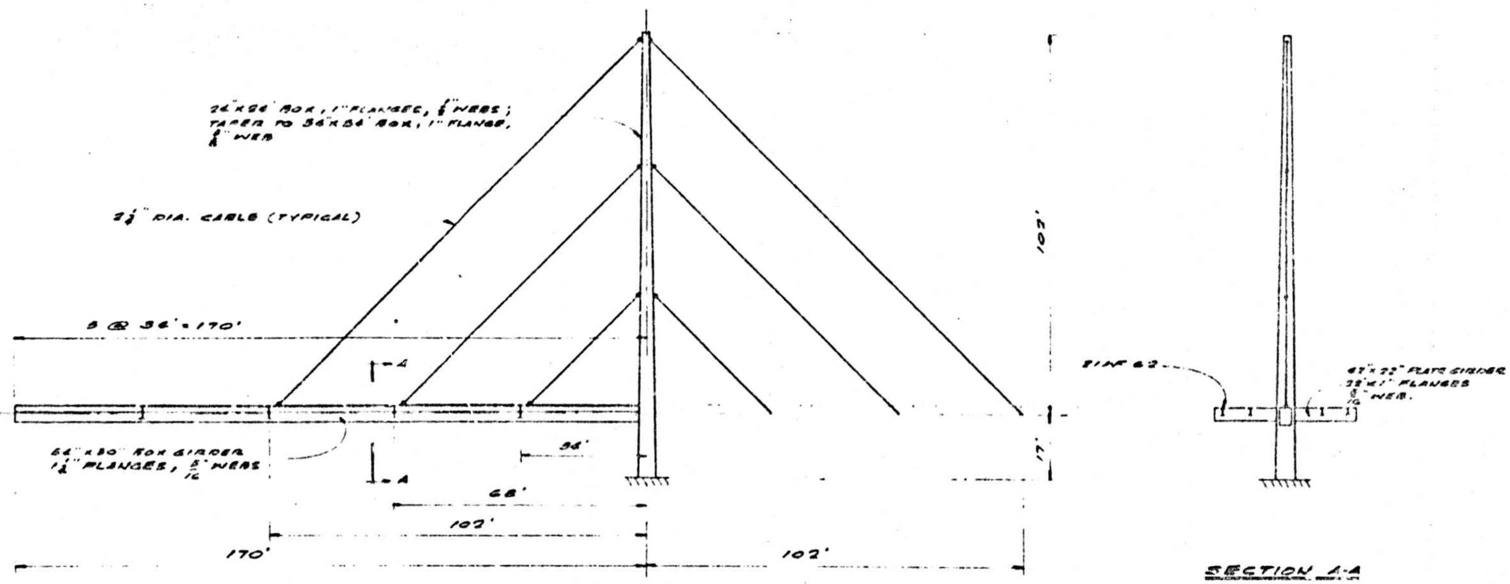


FIGURE 14. CONCEPT DESIGN SKETCH OF STAYED GIRDER BRIDGE CONCEPT IN NEW BRIDGE APPLICATION

2. Braced Arch Bridge

Inspired by a similar monorail bridge design in West Germany, this bridge consists of a single parabolic arch which springs from two "A" frames. The frames permit the roadway to pass directly beneath the arch. The roadway is suspended beneath the arch by cables. This aesthetically pleasing concept is best suited to site locations which provide natural foundations for the arch thrusts. Figure 15 illustrates the structural configuration employed in developing a conceptual design.

The Braced Arch Bridge is statically indeterminate and may be accurately analyzed by considering the flexibility of the arch and the cable ties, in that they provide elastic supports for the continuous girders. An analysis methodology which considers the total system is contained in Appendix B. In analyzing the structure for concept design purposes, however, a simplified analysis procedure was employed which considered the arch and continuous girder separately. Influence lines developed by considering the separate structures are presented in Figure 16. Design values developed from the influence lines for principal structural members are contained in Table XI. The concept design developed in Table XII and presented in Figure 17 provides an appraisal of the design of this bridge, although design procedures followed were not as detailed as those employed in developing preliminary designs of other concepts. A significant limitation determined during concept analysis and design concerns out-of-plane loading of the single arch by unsymmetrical live loads on the bridge deck. Effects of these types of loads on the arch structure were not completely evaluated in developing the concept design. Preliminary cost estimates were not developed for this structure because of the less comprehensive nature of the methodology employed in developing the concept design.

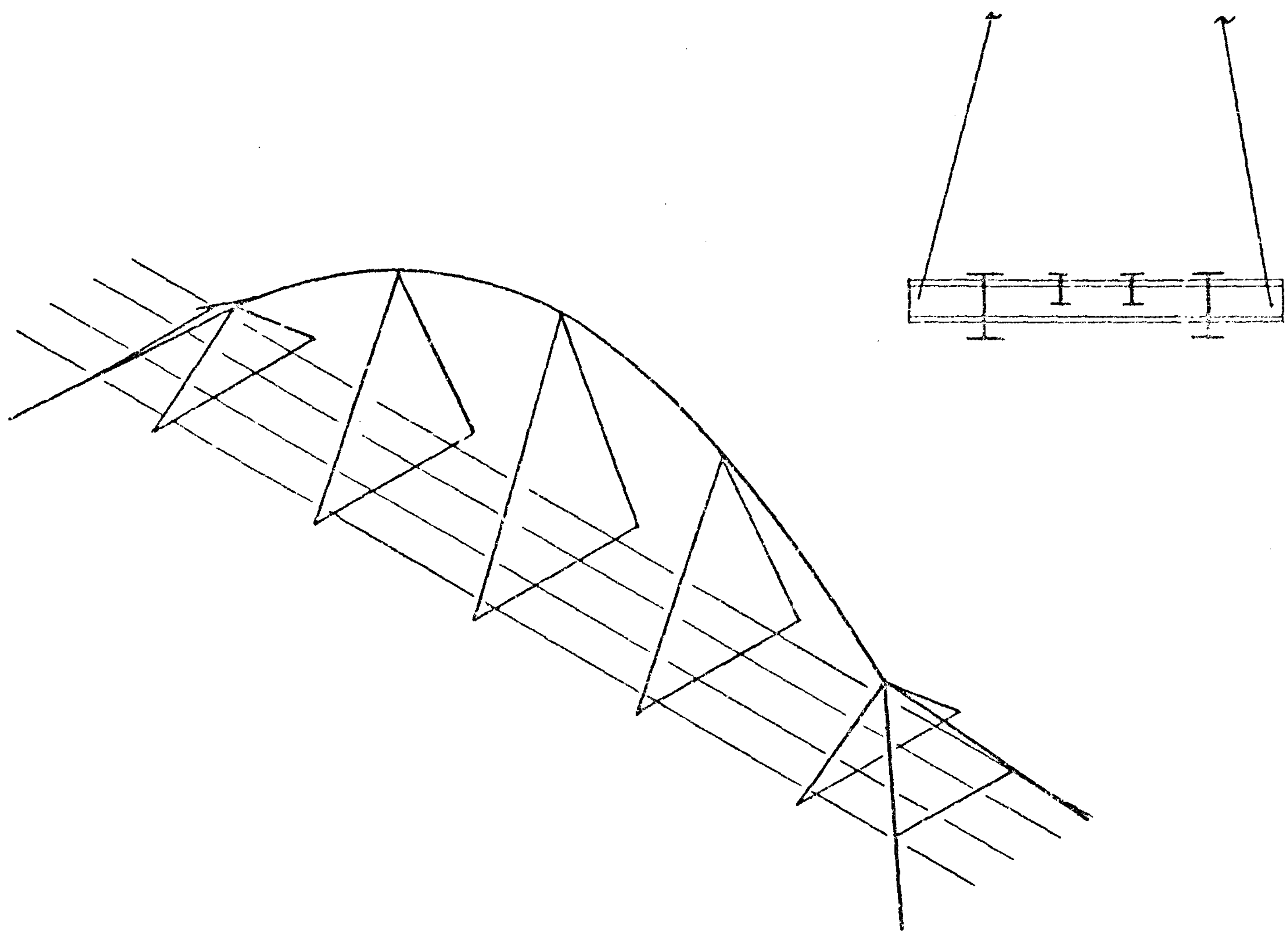


FIGURE 15. ISOMETRIC SCHEMATIC DRAWING OF BRACED ARCH BRIDGE CONCEPT IN NEW BRIDGE APPLICATION

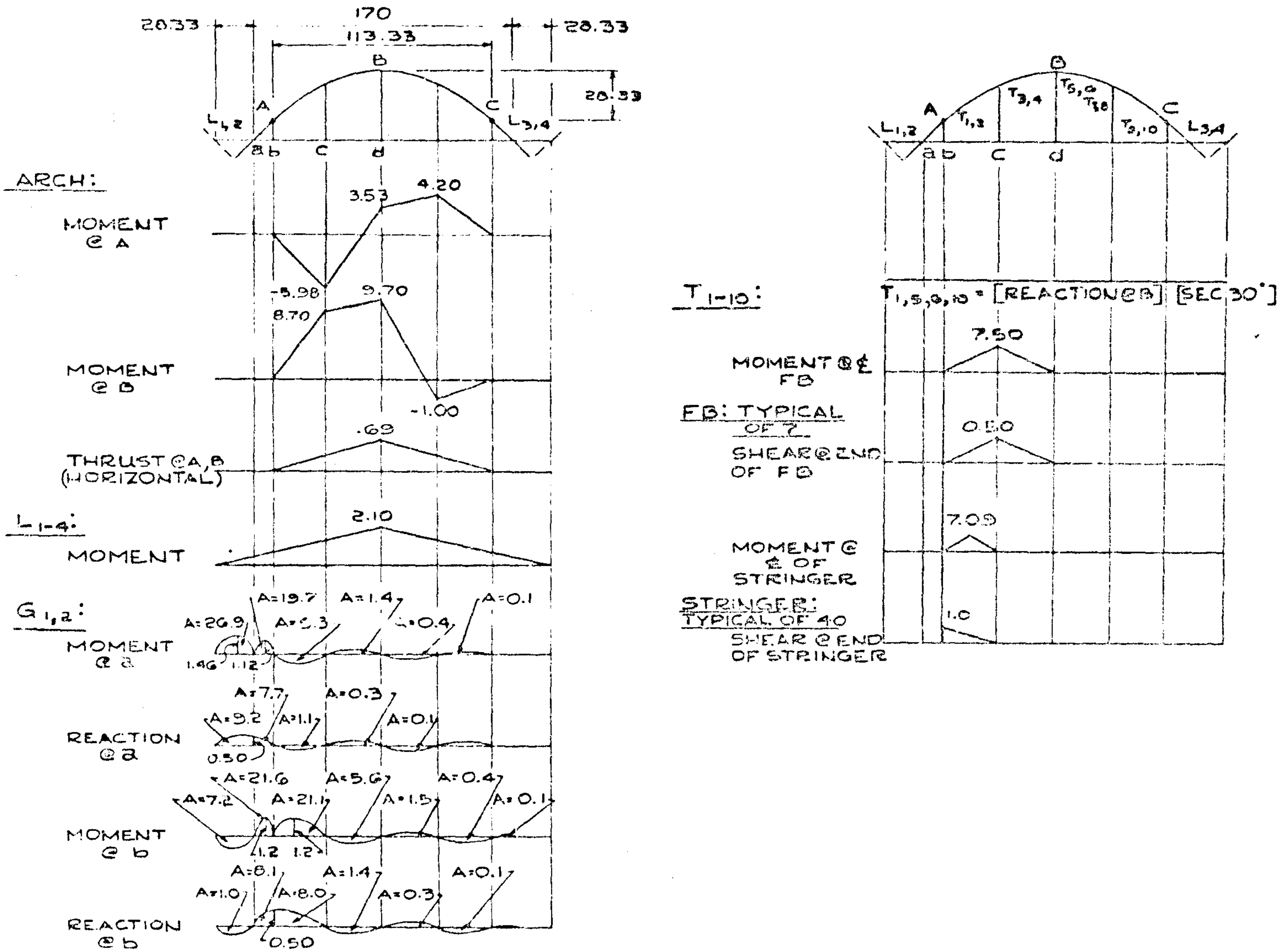


FIGURE 16 INFLUENCE DIAGRAMS FOR BRACED ARCH BRIDGE

TABLE XI. DESIGN VALUES: BRACED ARCH BRIDGE CONCEPT IN NEW BRIDGE APPLICATION

(a) Principal Structural Members

Member		Concentrated Loads: Equation for Concentrated Live Load Effect	Concentrated Live Load Effect	Impact	Concentrated Live Load Plus Impact	Uniform Loads: Equation for Uniform Load Effects	Dead Load Force Effect	Live Load Force Effect	Live Load Effect Plus Impact Effect	Total Effect for Design
Arch	Moments:									
	Crown:	M = 9.7P(1, 3)	349 K-ft	1.14(2)	398 K-ft	M_{DL} = 488w, M_{LL} = 503w	1470 K-ft	643 K-ft	733 K-ft	2601 K-ft
	Spring:	M = 4.2P(1, 3)	151 K-ft	1.14	172 K-ft	M_{DL} = 48.5w, M_{LL} = 187.5w	146 K-ft	240 K-ft	273 K-ft	591 K-ft
Horizontal	Thrusts:									
	Crown:	F = 0.69P(1)	24.8 K	1.14	28.3 K	F = 39.1w	117 K	50 K	57 K	192.3 K
	Spring:	F = 0.69P(1)	24.8 K	1.14	28.3 K	F = 39.1w	117 K	50 K	57 K	192.3 K
L_{1-4}	Axial Force:		see note (4)				see note (4)			153 K(4)
	Moments:		see note (4)				see note (4)			296 K-ft(4)
	Shear:	F = 0.50P	18 K	1.14	20.6 K	F = 226w	135.1 K	154.0 K	175 K	
$G_{1,2}$	Moment at a:	F = 1.46P(1)	52.5 K-ft	1.14	59.9 K-ft	DL: F = Σ w areas under influence dia = 42.4w(5)	127 K-ft			
						LL: F = Σ · areas under influence dia = 48.1w		61.5 K-ft	70.1 K-ft	257.0 K-ft (use for design)
	Moment at b:	F = 1.2P(1)	43.2 K-ft	1.14	49.5 K-ft	DL: F = Σ w areas under influence dia = 31.1w(5)	93 K-ft	56.6 K-ft	64.5 K-ft	207.0 K-ft
						LL: F = Σ · areas under influence dia = 44.3w(5)				
$T_{1,3,6,10}$	Axial Force at a:	F = 0.50 sec 30°P				DL: F = Σ w areas under influence dia = 13.9w(5)	41.8 K			
	Axial Force at b:	F = 0.57P	20.8 K	1.14	23.7 K	LL: F = Σ · areas under influence dia = 16.4w(5)		21.0 K	23.8 K	89.3 K (use for design of T_{1-10})

(1) P = 36 K as two lane concentrated loads effects.
(2) Span is 226 ft for impact factor computation.
(3) w_{DL} = 3000 plf (est) and w_{LL} = 1280 plf as loads for two lanes.
(4) Legs L_{1-4} supporting the arch are each assumed to carry 1/2 of the spring point arch moment, 1/2 sec 45° of the arch thrust.
(5) w_{DL} = 1200 plf (est) and w_{LL} = 1280 plf as uniform load for two lanes.

(b) FLOOR SYSTEM MEMBERS

(b) *Floor System Members*

Member		Concentrated Loads: Equation for Concentrated Live Load Effect	Concentrated Live Load Effect	Impact	Concentrated Live Load Plus Impact	Uniform Loads: Equation for Uniform Load Effects	Dead Load Force Effect	Live Load Force Effect	Live Load Effect Plus Impact Effect	Total Effect for Design
Floorbeams (typical of 7)	Moment:	F = 7.50P(1)	270 K-ft	1.28(2)	345 K-ft	$F = \frac{1}{2}$ (56.67)(7.50)w(3)	64.0 K-ft	272 K-ft	349 K-ft	1334 K-ft
	Shear:	F = 0.50(1)	18 K	1.28(2)	23.0 K	$F = \frac{1}{2}$ (56.67)(0.50)w(3)	42.5 K	18.2 K	23.4 K	88.9 K
Stringers (typical of 40)	Moment:	F = 7.09P(4)	42.5 K-ft	1.3(2)	55.2 K-ft	$F = \frac{1}{2}$ (28.33)(7.09)w(5)	75.0 K-ft	32.0 K-ft	41.5 K-ft	171.7 K-ft
	Shear:	F = 1.00P(4)	9 K	1.3(2)	11.7 K-ft	$F = \frac{1}{2}$ (28.33)(1.00)w(5)	10.5 K	4.5 K	5.8 K	72.0 K

(1) P is concentrated load for two lanes acting at center of floor beam, P = 36 K.
(2) Span = 56.67 for floorbeam impact computation, span = 28.33 for stringer impact computation (1.3 is maximum impact factor).
(3) w_{DL} = 3000 (est) and w_{LL} = 1280 as DL and LL for two lanes.
(4) P is equivalent wheel load on single stringer, assumed to be 6 K for moment 9 K for shear.
(5) w is equivalent to 1/2 of a lane load (6 ft of bridge width) for LL, 750 plf est DL and 320 plf for LL.

TABLE XII. CONCEPT DESIGN: BRACED ARCH BRIDGE CONCEPT IN NEW BRIDGE APPLICATION

Member	Design Value	Design Notes	Section	Area	Unit Weight	Length	Weight	Quantity	Total Weight	
Principal Structural Members										
Arch		Design tapered section								
Moments: Crown	2601 K-ft	Crown-S req'd = 1560 cu. in.	Crown: Boxgirder 60 X 30, 30 X 1-in. flanges, 5/16-in. webs	96.0 sq in.	244 plf avg					
Spring	591 K-ft	Spring-S req'd = 354 cu. in. Consider axial loads, increase S req'd	Spring: 30 X 15-in., 15 X 1-in. flanges, 5/16-in. webs.			129.8 ft	33.0 K	1	33.0 K	
Hor. Thrusts: Crown, Spring	192.3 K			47.5						
L_{1-4}: Moment, Axial	296 K-ft, 153 K	S req'd = 178 cu in.; use 30 WF 116 for estimate purposes considering axial stresses.	30 WF 116	--	116 plf	63.6	7.38 K	4	29.6 K	
$G_{1,2}$: Moment	257 K-ft	Design as constant section beam for maximum moment; S req'd = 154 cu in.	24 WF 76	--	76 plf	226.7 ft	17.2 K	2	34.4 K	
T_{1-10}: Axial Force	89.3 K	Tension members, allowable stress = 80 ksi, A req'd = 1.12 sq in.	1 1/2-in. dia cable	1.8 sq in.	6.1 plf	1, 2 30 ft	0.18	2	0.4 K	
						3, 4 50 ft	0.31	2	0.6 K	
						5, 6 59 ft	0.36	2	0.7 K	
						7, 8 50 ft	0.31	2	0.6 K	
						9, 10 30 ft	0.18	2	0.4 K	
						Total principal structural members			99.7 K	
						Miscellaneous (20%)			20.0 K	
										119.7 K
Floor System Members										
FB: Moment, Shear, Axial Force ($FB_{1,5,6,10}$)	1334 K-ft, 88.9 K, 44.0 K	Short deep beam with axial load, increase S by 10% to allow for column action. S req'd = 680 cu in.	36 WF 245	--	245 plf	50 ft	12.2 K	7	85.5 K	
Stringers: Moment, Shear	171.7 K-ft, 28.0 K	Use WF beam, S req'd = 103 cu in.	18 WF 60	--	60 plf	28.33 ft	1.7 K	32	54.5 K	
						Total floor system			140.0 K	
						Lateral bracing, etc. (15%)			21.0 K	
										161.0 K
						Total Bridge Weight				280.5 K

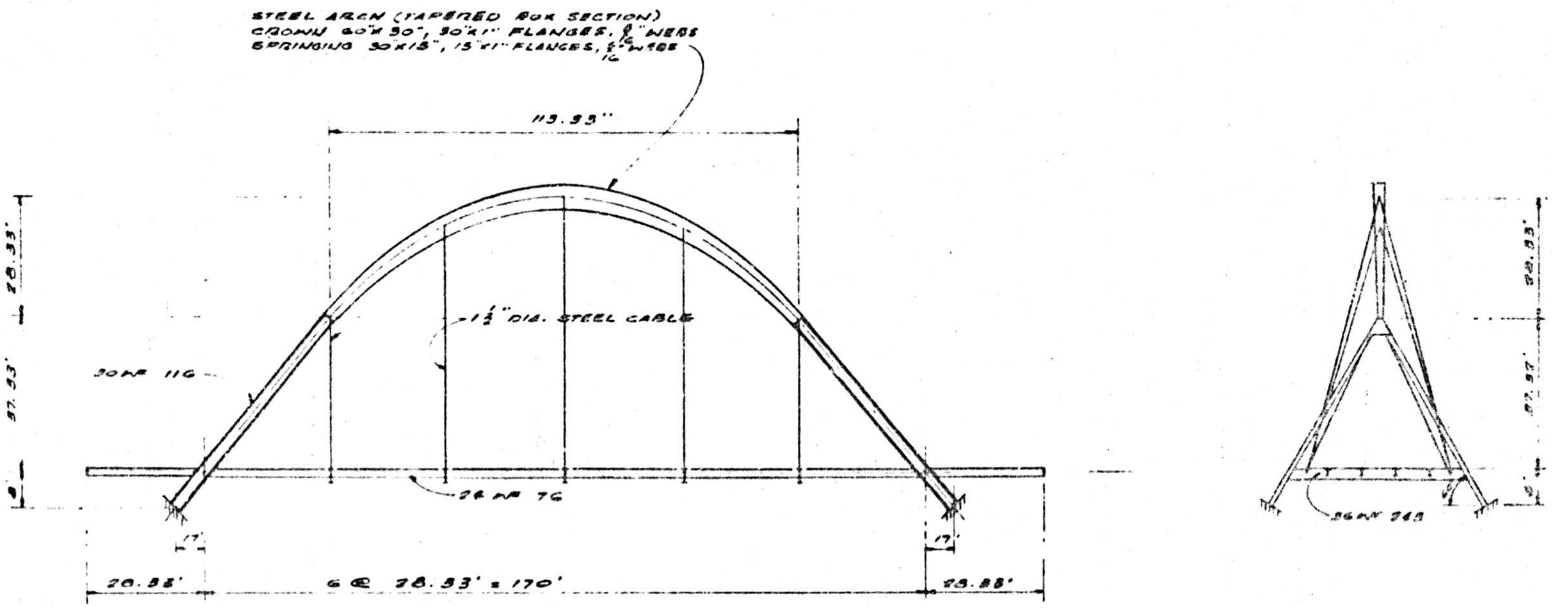


FIGURE 17. CONCEPT DESIGN SKETCH OF BRACED ARCH BRIDGE CONCEPT IN NEW BRIDGE APPLICATION

3. Leaning Piers Bridge

An attempt to acquire additional horizontal clearance by leaning supporting piers inward and suspending the center support from cables is presented in this bridge concept, illustrated in Figure 18. This concept is feasible for typical divided highway crossing structures; however, several disadvantages in the basic design are evident. There are no connections between the two girders and the leaning piers (to preclude large bending moments in the piers); therefore, the main girders bridge relatively long spans.

The structure is statically determinate and the influence lines presented in Figure 19 may be easily developed using the analysis methodology contained in Appendix B. Design values are computed and presented in Table XIII. Concept design considerations reflect the axial load situation which exists in the main girders, thus placing the procedure concerning design of these members into a beam-column type of analysis. The concept design developed for the Leaning Piers Bridge is summarized in Table XIV and illustrated in Figure 20. As in the case of the preceding two concept designs, the structure presented is not as refined as the preliminary designs illustrated for the leaning Arches and Bridle Bridges; however, an appraisal of the nature of a bridge design employing this feasible concept may be gained by reviewing the tabulated engineering data and drawings. Preliminary cost estimates were not developed for this structure because of the less comprehensive design methodology employed in developing the concept design.

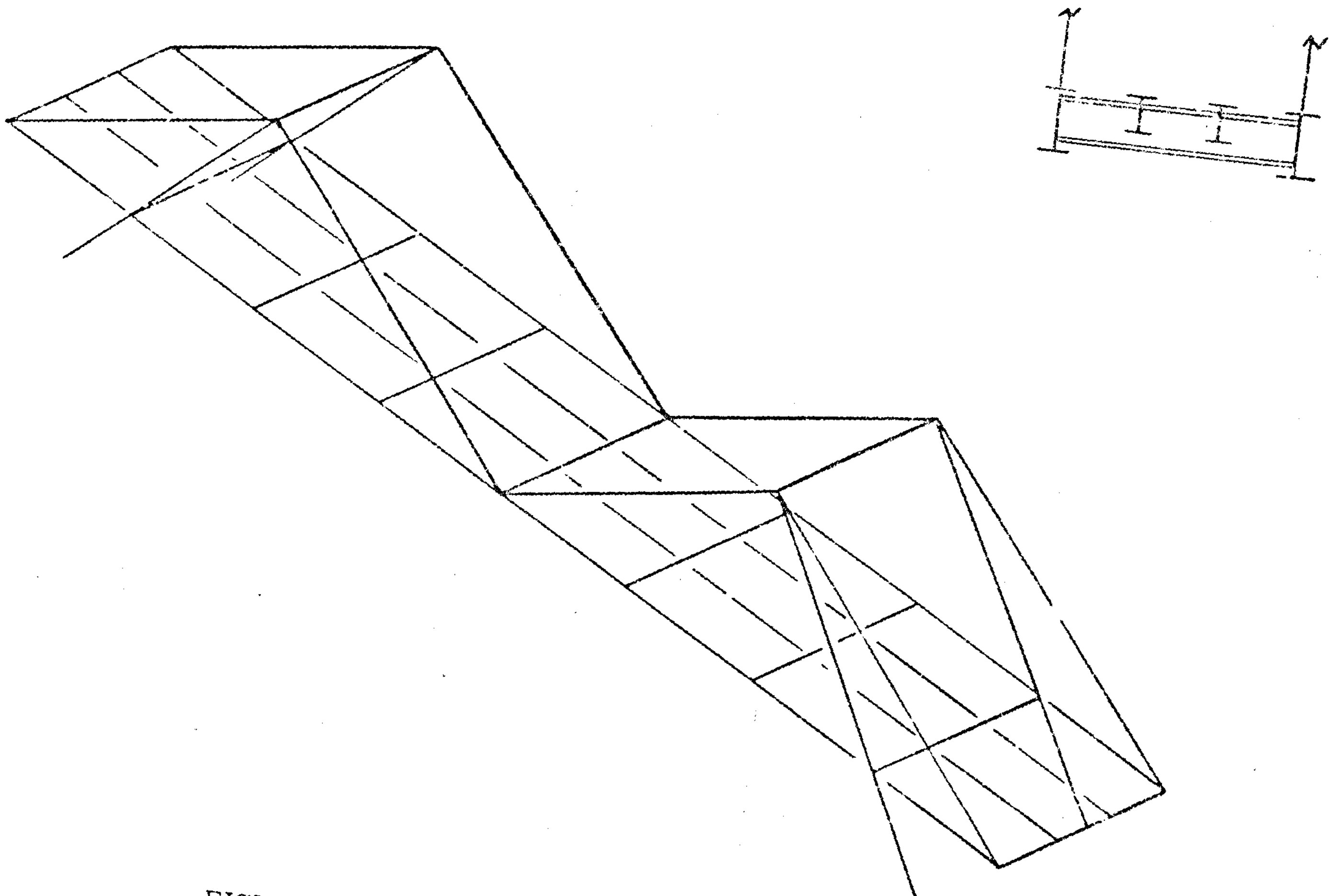


FIGURE 18. ISOMETRIC SCHEMATIC DRAWING OF LEANING PIERS BRIDGE IN NEW BRIDGE APPLICATION

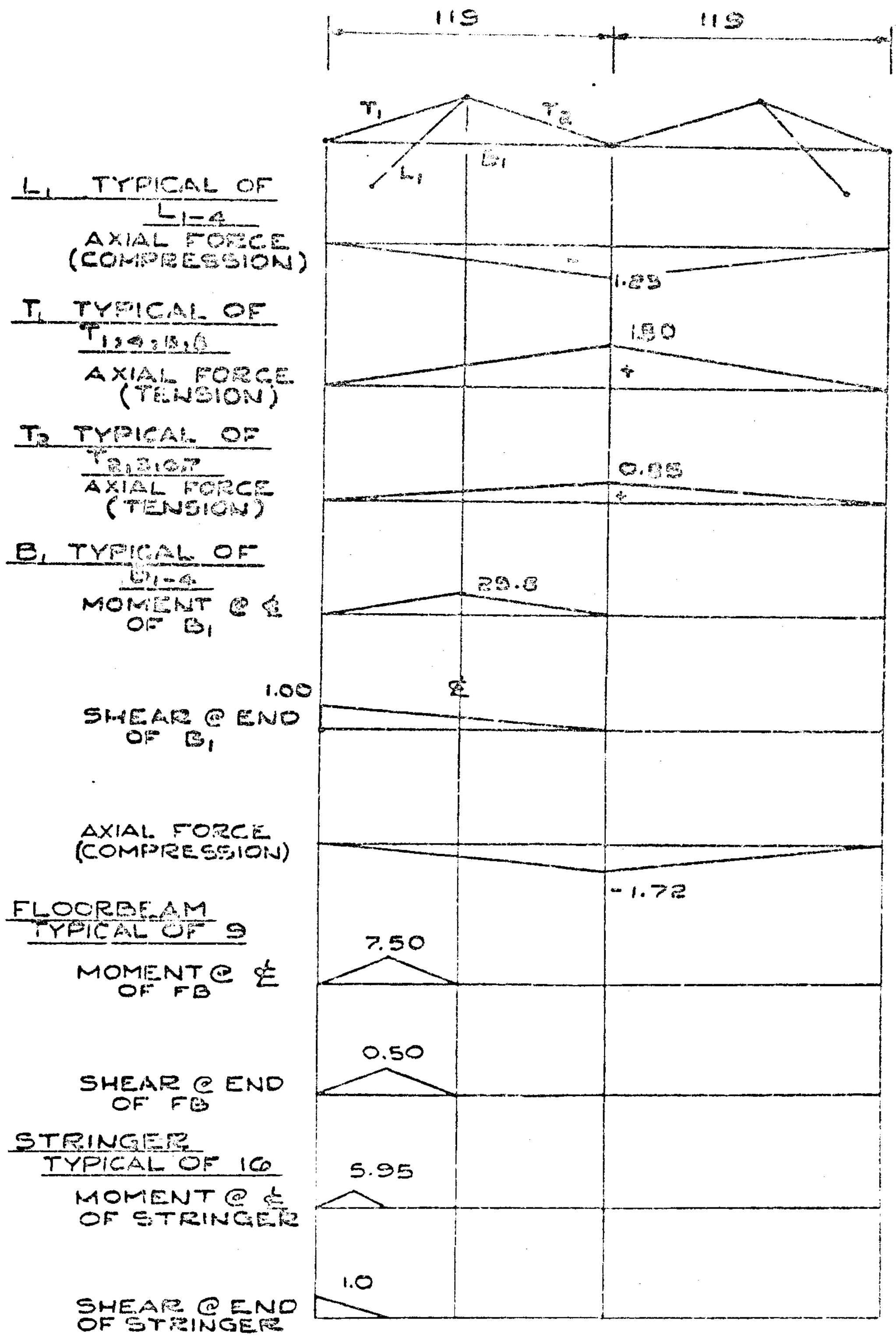


FIGURE 19 INFLUENCE DIAGRAMS FOR LEANING PIERS BRIDGE

TABLE XIII. DESIGN VALUES: LEANING PIERS BRIDGE CONCEPT IN NEW BRIDGE APPLICATION

(a) Principal Structural Members

Member	Concentrated Loads: Equation for Concentrated Live Load Effect	Concentrated Live Load Effect	Impact	Concentrated Live Load Plus Impact	Uniform Loads: Equation for Uniform Load Effect	Dead Load Force Effect	Live Load Force Effect	Live Load Effect Plus Impact Effect	Total Effect for Design
L	Axial Force: $F = -1.23p^{(1,2)}$	-22.3 K	$1.14^{(3)}$	-25.4 K	$F = -\frac{1}{2}(238)(1.23)w^{(4)}$	-220.0 K	-93.6 K	-106.8 K	-352.2 K
T_1	Axial Force: $F = +1.80p^{(1,2)}$	+32.4 K	$1.14^{(3)}$	37.0 K	$F = \frac{1}{2}(238)(1.80)w^{(4)}$	322.0 K	137.2 K	156.5 K	515.5 K
T_2	Axial Force: $F = +0.85p^{(1)}$	+15.3 K	$1.14^{(3)}$	17.4 K	$F = \frac{1}{2}(238)(0.85)w^{(4)}$	152.0 K	64.8 K	73.9 K	243.3 K
B	Axial Force: $F = -1.72p^{(1,2)}$	-31.0 K	$1.21^{(3)}$	-37.4 K	$F = -\frac{1}{2}(238)(1.72)w^{(4)}$	-305.0 K	-130.0 K	-148.0 K	-490.4 K
	Moment: $F = 29.8p^{(1,2)}$	536 K-ft	$1.21^{(3)}$	649 K-ft	$F = \frac{1}{2}(119)(29.8)w^{(4)}$	2650 K	1130 K-ft	1370 K-ft	4669 K-ft
	Shear: $F = p^{(2,5)}$	26 K	$1.14^{(3)}$	29.6 K-ft	$F = \frac{1}{2}(119)(1.00)w^{(4)}$	89.2 K	38.1 K-ft	46.0 K	164.8 K

(1) Tension is +, compression is -.
(2) P = 18 K, 18 K concentrated load in each of two lanes acting at bridge centerline (K is Kips).
(3) Span = 238 ft for principal structural members.
(4) w_{DL} = 1500 plf for each lane, w_{LL} = 640 plf for each lane.
(5) Concentrated load for shear = 26 K per lane.

(b) Floor System Members

Member	Concentrated Loads: Equation for Concentrated Live Load Effect	Concentrated Live Load Effect	Impact	Concentrated Live Load Plus Impact	Uniform Loads: Equation for Uniform Load Effect	Dead Load Force Effect	Live Load Force Effect	Live Load Effect Plus Impact Effect	Total Effect for Design
Floor-beams	Moment: $F = 7.50P^{(1)}$	270 K-ft	$1.29^{(2)}$	351 K-ft	$F = \frac{1}{2}(47.6)(7.50)w^{(3)}$	538 K-ft	226 K-ft	294 K-ft	1183 K-ft
	Shear: $F = 0.50P^{(1)}$	16 K	$1.29^{(2)}$	22.4 K	$F = \frac{1}{2}(47.6)(0.50)w^{(3)}$	35.6 K	15.2 K	19.6 K	77.6 K
Stringers	Moment: $F = 5.95P^{(4)}$	35.7 K-ft	$1.3^{(2)}$	46.2 K-ft	$F = \frac{1}{2}(23.8)(5.95)w^{(5)}$	53.0 K-ft	22.6 K-ft	29.4 K-ft	128.6 K-ft
	Shear: $F = 1.00P^{(4)}$	9 K	$1.3^{(2)}$	11.7 K	$F = \frac{1}{2}(23.8)(1.0)w^{(5)}$	9.0 K	3.8 K	5.0 K	25.7 K

(1) P is equivalent concentrated live load for two lanes acting at center of floor beam span; P = 36 K.
(2) Span = 47.6 for floorbeam impact computation; span = 23.8 for stringer impact computation (max impact factor = 1.3).
(3) w_{DL} = 3000 and w_{LL} = 1280 plf as est dead load and live load for two lanes.
(4) P is equivalent wheel load on single stringer, assumed to be 6 K for moment, 9 K for shear.
(5) w is equivalent to 1/2 of a lane load (6 ft of bridge width) for live load; 750 plf est for dead load and 320 plf for live load.

TABLE XIV. CONCEPT DESIGN LEANING PIERS BRIDGE CONCEPT IN NEW BRIDGE APPLICATION

Member	Design Value	Design Notes	Section	Area	Unit Weight	Length	Weight	Quantity	Total Weights	
Principal Structural Members										
L_1: Axial Force	-352.2 K	Design as column, L = 52.4, radius of gyration approximately 6 inches. Box design indicated: allowable stress = 10 ksi, r = 6 in.	18 × 16 × 1/2 in. square box column	35 sq in.	119 plf	52.4 ft	6.23 K	4 K	25.0 K	
T_1: Axial Force	515.5 K	Tensile member, use cable with allowable stress = 80 ksi A req'd = 6.5 sq in., use 2-7/8-dia.	2-7/8-in. dia. cable	6.5 sq in.	22.1 plf	58 ft	1.28 K	4	5.1 K	
T_2: Axial Force	243.3 K	A req'd = 3.04 sq in.	1-5/8-in. dia. cable	2.1 sq in.	7.1 plf	68 ft	0.49 K	4	2.0 K	
B_1: Axial Force / Moment / Shear	-490.4 K / 4669 K-ft / 144.8 K	Design as plate girder, S req'd = 2620 cu in.; consider axial stresses.	78 × 24 in., 24 × 2 in. flanges, 3/8-in. web	123.6 sq in.	420.0 plf	119 ft	50.0 K	4	200.0 K	
						Total principal structure			232.1 K	
						Bracing, stiffeners, etc. (15%)			34.8	266.9 K
Floor System M	£t									
FB: Moment / Shear	1183 K-ft / 77.6 K	Short deep beam, S req'd = 710 cu in., use WF beam	36 WF 230	--	230 plf	30 ft	6.9 K	9	62.1	
Stringers: Moment / Shear	128.6 K-ft / 25.7 K	Use WF beam, S req'd = 77.0 cu in.	16 WF 50	--	50 plf	23.8 ft	1.19 K	40	47.7	
						Total floor system			109.8	
						Diaphragms, stiffeners, etc. (15%)			16.5	126.3
						Total Bridge Weight				393.2

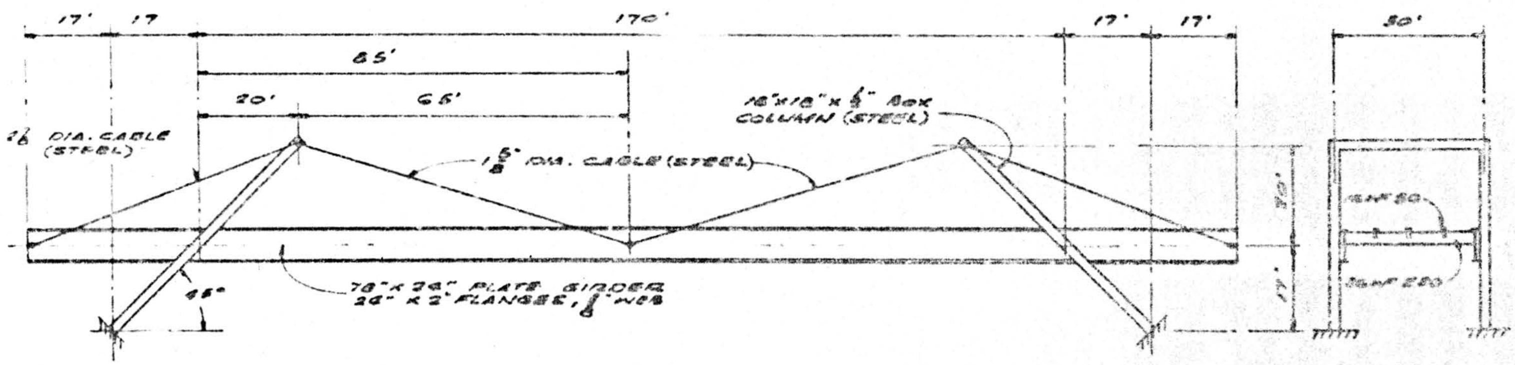


FIGURE 20. CONCEPT DESIGN SKETCH OF LEANING PIERS BRIDGE CONCEPT IN NEW BRIDGE APPLICATION

IV. PRELIMINARY DESIGNS FOR MODIFYING EXISTING BRIDGES

Two new bridge design concepts were judged to be feasible in applications involving modification of existing bridges to permit removal of massive support structures. These are: (1) the Leaning Arches Bridge, and (2) the Frame Bridge. There are many types of existing structures which could be modified to effect removal of abutments and median piers, and each of these structures possesses its own unique geometric and structural characteristics. Therefore, it would be difficult to conjecture concerning "typical" or "standard" situations for applications effectiveness studies employing new bridge concepts. In this section of the report, the two bridge concepts identified above are applied to bridge situations that have been determined to exist in practice. The resulting preliminary analysis and design exercise is intended to illustrate the probable design and construction process that would be followed if a decision were made to modify an existing bridge of the type selected. In so doing, the specific illustration should provide the highway engineer with a general appraisal of the processes involved, including an understanding of the required degree of departure from conventional design and construction practices.

In developing the preliminary designs, the safety-related geometric criteria discussed in Section II were employed in conjunction with an assumed 0° skew crossing structure to establish the gross geometrics of the bridge situation to be modified. Restrictions as to the types of existing bridge that may be modified by employing the concepts are noted in discussing specific concept applications in subsequent paragraphs. If extensive bridge modification efforts are warranted by a review of accident history and safety considerations, an inventory of existing bridge structures should be made and bridges should be categorized according to basic types (structures with same span ratios, superstructure design, etc.). From these groupings, analyses could be conducted to indicate the relative design effectivenesses of the several feasible bridge concepts.

The existing bridge modification construction sequence consists of (1) temporarily shoring the existing bridge floor system, (2) subsequent removal of the median pier and/or abutments, and (3) installation of transverse floor beams suspended from the new structure by cables, thus providing the floor systems with support previously furnished by the interior bents. Conventional structural steel design practice and fabrication methods are consistent with these construction concepts.

A. Modification of Existing Bridge Employing Leaning Arches Bridge Concept

The Leaning Arches Bridge concept applied to modification of existing structures differs from the Leaning Arches Bridge described in the new structures application section (Section III) only in the number of cables employed. A system of parallel cables suspended from the arches (in the plane of the arches) is used to support the existing floor system, thus allowing removal of median pier and abutment supports. The existing bridge may be of continuous or simple span design.

1. Application Discussion

The Leaning Arches Bridge concept provides a great deal of flexibility in selecting a support configuration for the existing structure. This concept applied to an existing four span, simple or continuous, bridge is illustrated in Figure 21. The designer may choose to support the structure with the same span relationship employed in the existing bridge by replacing the remov d interior bent caps with transverse floor beams, or he may find it necessary to provide additional, intermediate floor beams. The flexibilities of the arches and cables must be considered in selecting the number and spacing of floor system supports.

2. Analysis Discussion

Since the arch geometry chosen for the Leaning Arches Bridge employed in a modification configuration is similar to that employed in a new configuration, the influence lines are similar for the two structures. The moment of inertia of the arch section affects the influence line values, but, for the chosen conditions of constant moment of inertia throughout the arch lengths, very little difference in influence line values between the two applications are experienced. Significant changes in these values would be noted, however, if the moment of inertia of the arch varied accordingly along the length of the arch, or if the basic geometry of the arch were changed. For the structure pictured schematically in Figure 22, a constant moment of inertia was selected. The influence lines, for the principal structural members, therefore, are contained in Figure 4.

3. Preliminary Design Discussion

Design values were developed for principal structure components of the modified bridge using the influence lines presented in Figure 4. These design values are presented in Table XV. A preliminary design of the structure was developed from these design values and is summarized in Table XVI. The arches are steel box sections with constant cross section. The transverse floor beams are conventional plate girder construction. The preliminary design drawing for this concept is shown in Figure 23.

Although the existing stringers are shown resting on top of the new floor beams, integral construction could be considered. In this particular design, the existing bridge was assumed to consist of four, 55-ft 0-in. continuous or simple spans, with cables in the plane of the arch used to support floor beams at each former interior bent location. If necessary, additional cables could be employed to provide additional supports.

The Leaning Arches Bridge is an attractive cable-supported bridge concept which may be employed in new and modified existing bridge construction. The concept provides the designer with many options. The solution to indeterminate structures problems can be accelerated with the use of computers to develop optimum structural configurations. The construction methods do not deviate substantially from conventional methods, and current cost estimating practices should be applicable.

4. Cost Discussion

Preliminary cost estimates were prepared for the modification of an existing bridge utilizing the Leaning Arches Bridge concept (Table XVII). The cost estimates were based on the modification situation shown in Table XVII and Figure 23. By shoring the existing structure and removing the three interior bents, the existing deck and stringer system may be salvaged. Unit costs for items 1 and 4 in the estimate were taken from current* Texas Highway Department bid averages; unit costs for items 2 and 3 are provided for both prototype and subsequent procurement in quantity construction. The lump sum cost of removing the interior bents was estimated from recent experience with a similar situation. Since it is desirable to minimize the hazardous situation created during actual bridge modification operations, a lump sum for a warning system has been included in the cost estimate.

*First quarter, calendar year 1969.

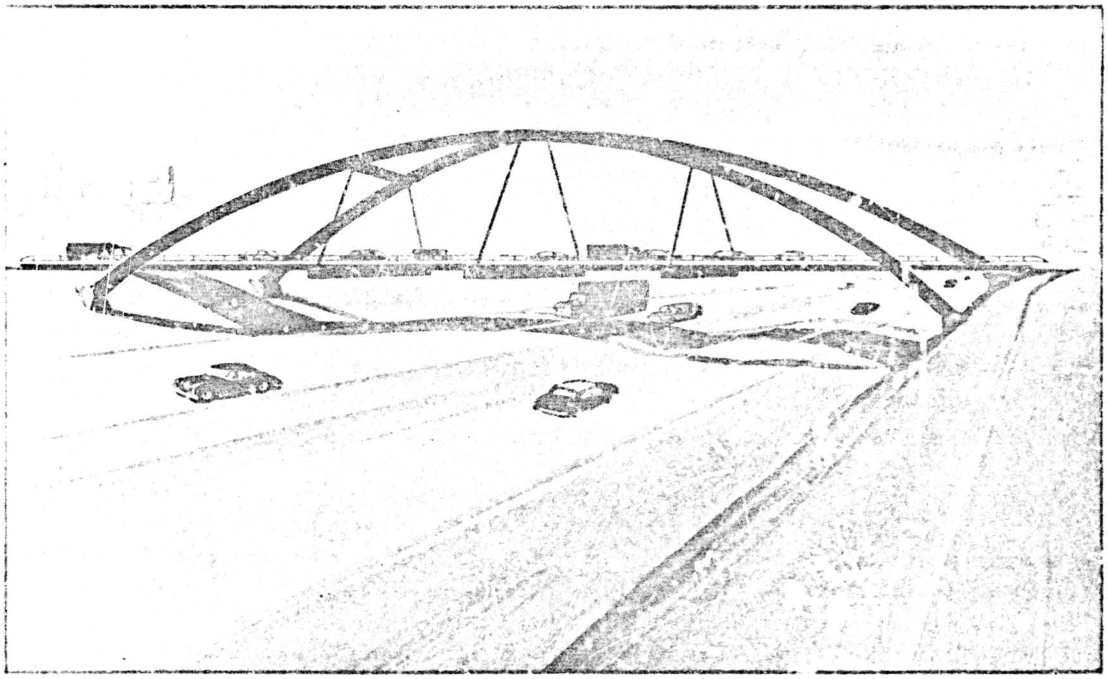


FIGURE 21. ARTIST'S SKETCH OF LEANING ARCHES BRIDGE CONCEPT IN MODIFIED BRIDGE APPLICATION

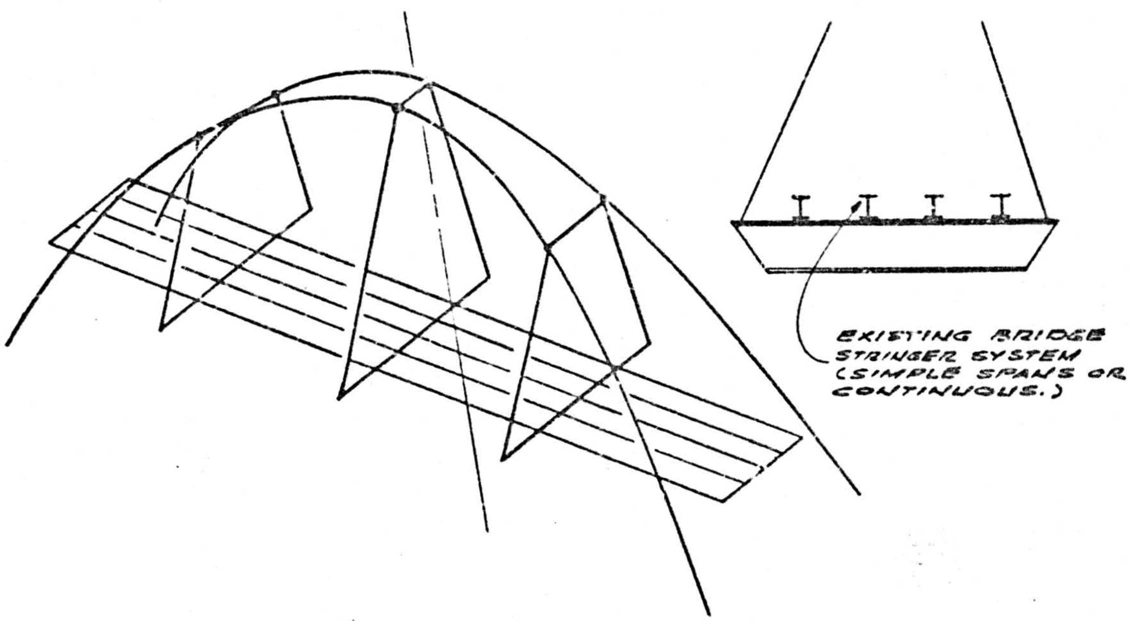


FIGURE 22. ISOMETRIC SCHEMATIC DRAWING OF LEANING ARCHES BRIDGE CONCEPT IN MODIFIED BRIDGE APPLICATION

TABLE XV. DESIGN VALUES: LEANING ARCHES BRIDGE CONCEPT IN MODIFIED BRIDGE APPLICATION (SUPERSTRUCTURE MEMBERS)

	Concentrated Loads				Uniform Loads				
Member	Equation for Concentrated Live Load Effect	Concentrated Live Load Effect	Impact	Concentrated Live Load Plus Impact	Equation for Uniform Load Effect	Dead Load Force Effect	Live Load Force Effect	Live Load Effect Plus Impact Effect	Total Effect for Design
Arch	Moments								
	$M_{max} = 7.5P^{(1)}$	270 K-ft	1.27	343 K-ft	$M_{DL} = 17w^{(4)}$ $M_{LL} = 343w$	522 K-ft	442 K-ft	560 K-ft	1425 K-ft
	Thrust								
	$F_{max} = 0.43P^{(1)}$	15.5 K	1.27	19.[illegible] K	$F = 55.5w$	167 K	71 K	90 K	276 K
Tension Members									
	Axial Load								
	$T_1 = T_2 = T_3 = 0.61P$	22 K	1.27	27.9 K	$F = 33.9w$	101.5 K	43.3 K	54.9 K	164.3 K
Floorbeam									
	Moment								
	$M = 10.8P$	389 K-ft	1.27	470 K-ft	$M = 594w$	1660 K-ft	760 K-ft	920 K-ft	3050 K-ft
	Shear								
	$V = 0.5P$	18 K	1.27	21.8 K	$V = 27.5w$	77 K	35.2 K	42.6 K	141.4 K

(1) P = 36 K as concentrated load from two lanes.
(2) Arch moment and thrust equations obtained from method in Borg and Gennaro, Advanced Structural Analysis, D. Van Nostrand Co Inc, 1959.
(3) Span for impact computation = 55 ft.
(4) w_{DL} = 1200 plf and w_{LL} = 1280 plf as uniform loads for two lanes.

TABLE XVI. PRELIMINARY DESIGN: LEANING ARCHES BRIDGE CONCEPT IN MODIFIED BRIDGE APPLICATION (SUPERSTRUCTURE MEMBERS)

Member	Design Value	Design Notes	Section	Area	Unit Weight	Length	Weight	Quantity	Total Weight
Principal Structural Members									
Arches Moment	M_{max} = 1425 K-ft			77 sq in.	262 plf	250 ft	65.5 K	2	131.0 K
Thrust	F_{max} = 276 K	Design as two box girders leaning inward and joining at crown.	Box girder 48 in. × 24 in. 1-in. flanges, 5/16-in. web						
Tension Members									
T_1:						30 ft	1.0 K	2	2.0 K
T_2:	164.3 K	Design as tension member for 164 K load, allowable stress 20 ksi A req'd = 9.2 sq in.	Use 3-1/2-in. dia cable	9.6 sq in.	32.6 plf	47 ft	1.53 K	2	3.1 K
T_3:						30 ft	1.0 K	2	2.0 K
Floorbeam									
Moment	3050 K-ft	Plate girder design indicated, S req'd = 1830 cu in.	60 × 20 in., 20 × 1 3/4-in. flanges, 5/16-in. web	117.7 sq in.	400 plf	58 ft	23.2 K	3	69.6 K
Shear	141.4 K					Total Main Structure			207.7 K
						Miscellaneous (15%)			31.0 K
						Total Bridge Weight			238.7 K

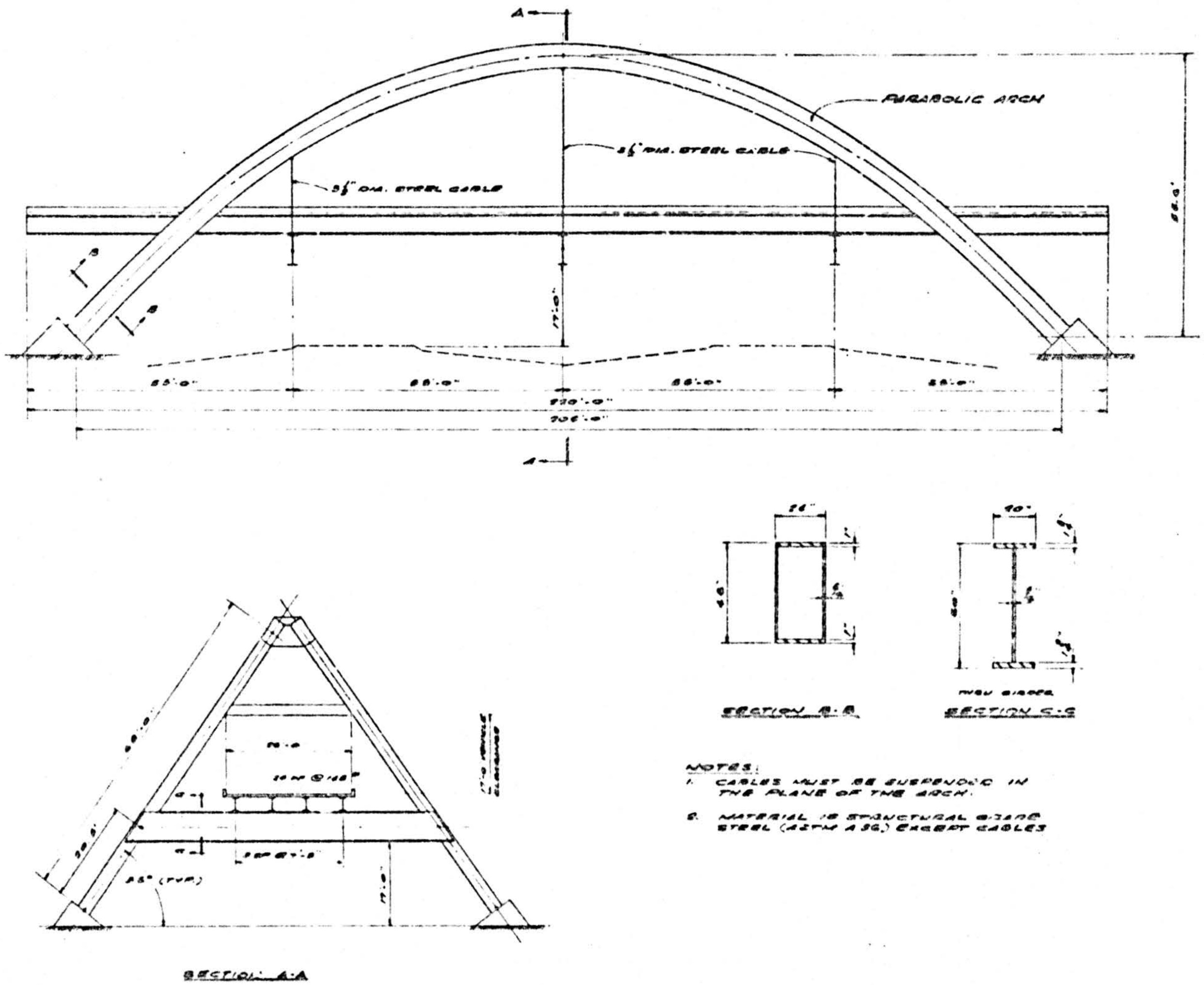
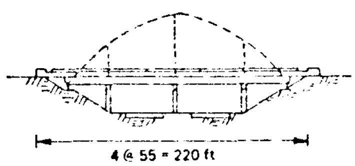


FIGURE 23. PRELIMINARY DESIGN DRAWING OF LEANING ARCHES BRIDGE CONCEPT IN MODIFIED BRIDGE APPLICATION

TABLE XVII. PRELIMINARY COST ESTIMATES, LEANING ARCHES BRIDGE IN MODIFIED BRIDGE APPLICATION

Item[1]	Unit	Quantity[2]	Unit Cost[3]	Cost Prototype Only	Cost Quantity
1. Structural Steel, Girder	lb	69,600	$0.25	$ 17,400	
2. Structural Steel, Fab. (Arch)	lb	131,000	0.50(0.30)[4]	65,500	(39,200)
3. Steel Cable	lb	7,100	0.75(0.65)[4]	5,320	(4,620)
4. Misc. Steel	lb	31,000	0.30	9,300	
5. Remove Interior Bents (3 ea)	ls	-	-	6,000[5]	
6. Barricades and Warning System	ls	-	-	1,000[5]	
Total Estimated Cost (Bridge Modification, excl arch substructures)				$104,520	($77,520)

(1) Stringer, deck and rail system salvaged intact from original bridge.
(2) Items 1 - 4 from Table XVI.
(3) Items 1 and 4 from first quarter (CY 69) Texas Highway Department construction cost data for in place units shown. These components are conventional bridge construction items.
(4) Items 2 and 3 are estimated in place unit costs for one-of-a-kind bridge installation. Procurement in quantity (unit and total costs) are shown in parentheses.
(5) Lump sum estimates based on modification of four span continuous bridge as illustrated below:



B. Frame Bridge Concept

Perhaps the most obvious answer to the requirement for a bridge superstructure which will allow removal of a median pier from an existing bridge is some form of a rigid frame positioned over the existing structure, with cables supporting floor beams which replace interior bent caps. The structural scheme which evolved from concept evaluations of frame type bridges consists of a three-dimensional rigid frame which straddles the crossing roadway in the manner illustrated in Figure 24. Cables connected to the frame at the intersection of the inclined and horizontal components support floor beams located at the floor system support points previously occupied by the interior bent caps. The behavior of the structure under live load requires that the floor system girders carry a nominal axial force; therefore, application of this concept is limited to existing structures with continuous girder systems.

1. Application Discussion

This bridge concept provides a relatively straightforward method for modifying an existing continuous structure which does not meet the safety-related geometric requirements due to the presence of a median pier and interior bents adjacent to the roadway. The modification sequence consists of (1) temporarily shoring the existing bridge, (2) removing the hazardous supports, (3) providing new floor beams, and (4) suspending these new beams from the new rigid frame structure, as shown in Figure 24. The new floor beams provide the support previously furnished by the interior bent caps.

2. Analysis Discussion

The three-dimensional rigid frame is a statically indeterminate structure which is loaded by the cables; a method for analyzing the indeterminate structure representing this concept is included in Appendix B. A computerized solution, employing the analysis method, was developed to analyze a Frame Bridge structure configured as shown in Figure 24. The computer program is presented in Appendix D. Although the output of this computer program provides influence line values for principal structural members, a generalized computer solution incorporating AASHO Specification loadings could produce a complete spectrum of design moments, shears, axial loads, and deflections. Such a program should be capable of handling unsymmetrical structures with a variety of spans and span ratios. Influence lines were plotted from computer output data; these influence lines are presented in Figure 25. These data were used to compute the design values presented in Table XVII.

3. Preliminary Design Discussion

Using the design values from Table XVII, a preliminary structural design was developed. Table XVIII contains a summary of the preliminary design, and Figure 26 is a design sketch of the Frame Bridge structure applied to an existing continuous plate girder bridge. This preliminary design is based on the use of an existing continuous steel I beam span floor system with the rigid frame/cable support superstructure. The rigid frame is constructed of steel plate configured as a box girder. The inclined and horizontal members were designed for maximum moment at their intersections. By employing tapered sections and plate thickness transitions, significant weight savings could be accomplished; however, a comprehensive design optimization program would be required. The exterior transverse floor beams are conventional steel plate girders which not only react vertical loads entering through the stringers, but also carry horizontal components of the cable forces. The girder section required to resist bending loads is sufficient to also resist cable component forces. This may not be the case for other frame and cable geometries. The center floor beam is a steel box section. The box section was chosen to react the bending loads caused by force unbalances in the cable horizontal components created by unsymmetrical placement of live loads.

4. Cost Discussion

Preliminary cost estimates were prepared for the modification of an existing bridge using the Frame Bridge concept (Table XX). The existing bridge structure assumed for the purposes of the estimate is shown in Table XX; this existing bridge is identical to the one used for the Leaning Arches Bridge cost analysis (Table XVII). Unit prices developed from current Texas Highway Department bid averages* were used to estimate costs for conventional construction items, while unit costs for the frame in the prototype estimate reflect the nonconventional nature of this item. The cost of the frame, if the Frame Bridge were to be constructed in quantity, computed by using conventional unit prices for fabricated steel, is also presented (shown in parentheses). The costs for removing the three interior bents were estimated as a lump sum based on a recent, similar project experience. The estimate also includes the cost of an adequate warning system to be used during construction.

The materials quantities for the Frame Bridge preliminary cost estimate are based on the preliminary design (Table XIX). The Frame Bridge preliminary design does not reflect the effects of a weight optimization study. It is possible that significant weight and, therefore, cost savings

*First quarter, calendar year 1969.

could be effected through such an effort, although it is doubtful that this concept could be made competitive with the Leaning Arches Bridge concept applied to modified bridge construction.

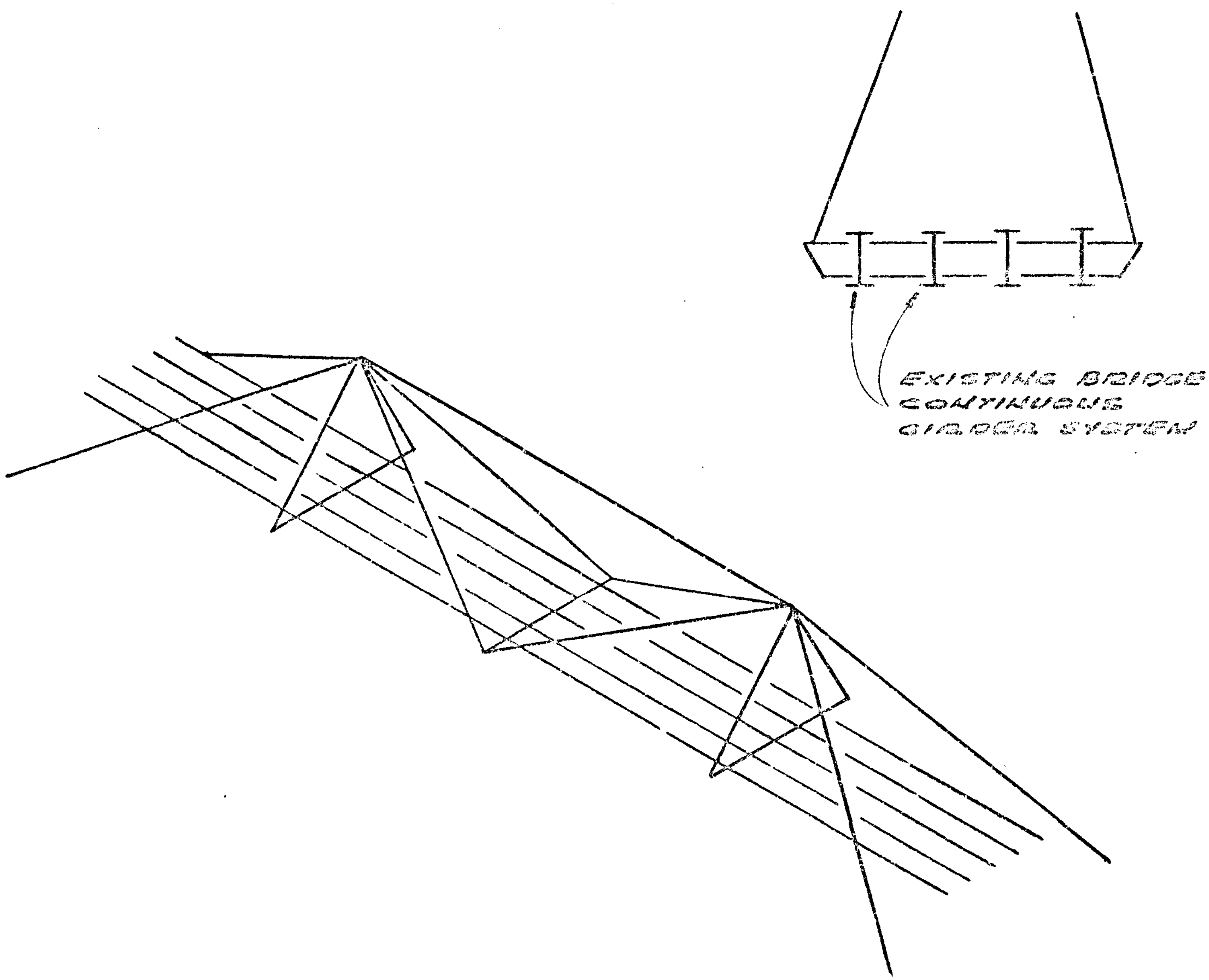


FIGURE 24. ISOMETRIC SCHEMATIC DRAWING OF FRAME BRIDGE CONCEPT IN MODIFIED BRIDGE APPLICATION

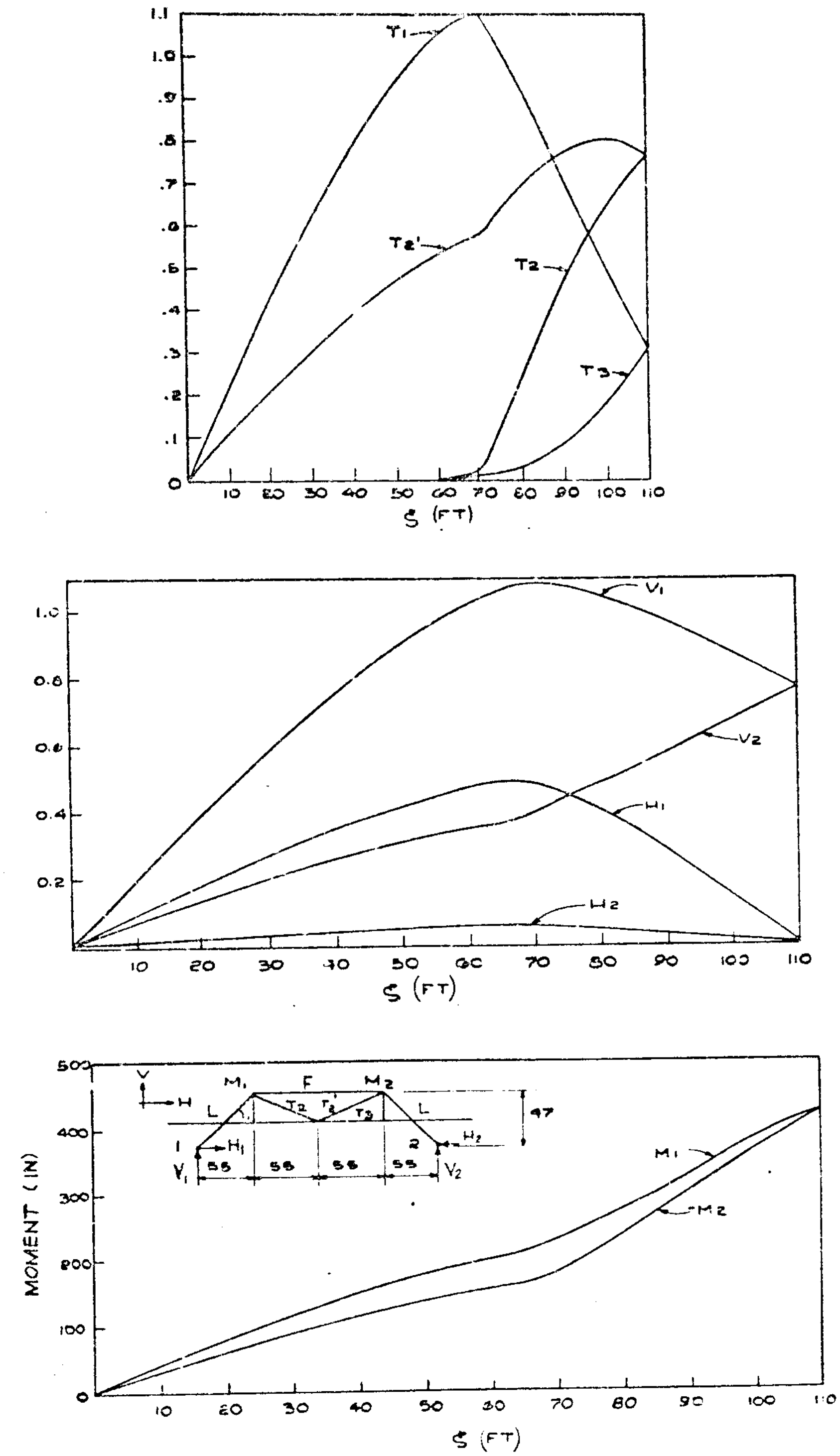


FIGURE 25. INFLUENCE DIAGRAMS FOR FRAME BRIDGE

TABLE XVIII. DESIGN VALUES: FRAME BRIDGE CONCEPT IN MODIFIED BRIDGE APPLICATION (SUPERSTRUCTURE MEMBERS)

	Concentrated Loads				Uniform Loads				
Member	Equation for Concentrated Live Load Effect	Concentrated Live Load Effect	Impact	Concentrated Live Load Plus Impact	Equation for Uniform Load Effect	Dead Load Force Effect	Live Load Force Effect	Live Load Effect Plus Impact Effect	Total Effects for Design
Tension									
$T_{1,3}$	$F = 0.550P$[1]	19.8 K	1.21[2]	24.0 K	$F = 39.6w$[3]	142.5 K	50.5 K	61.0 K	227.5 K
$T_{2,2}$	$F = 0.405P$	14.6 K	1.21	17.7 K	$F = 35.2w$	126.5 K	45.0 K	54.5 K	198.7 K
Legs									
L	Axial Force: $F = 0.313P$	11.3 K	1.15[4]	13.0 K	$F = 58.2w$	209.0 K	74.5 K	85.7 K	307.7 K
	Moment: $M = 18.0P$	648 K-ft	1.15	747 K-ft	$M = 1660w$	5970 K-ft	2120 K-ft	2440 K-ft	8157 K-ft
	Shear: $F = 0.225P$	8.1 K	1.15	9.3 K	$V = 20.7w$	74.5 K	26.5 K	30.5 K	114.3 K
F									
	Moment: $F = 25.2P$	907 K-ft	1.15	104 K-ft	$M = 2250w$	8100 K-ft	2880 K-ft	3315 K-ft	12,450 K-ft
	Axial Force: $F = 0.255P$	9.2 K	1.15	10.6 K	$F = 90.6w$	337.0 K	116.0 K	133 K	480.6 K
Floorbeam (exterior)	Moment: $M = 10.6P$[1]	382 K-ft	1.21[2]	462 K ft	$M_{DL} = 665w_{DL}$ $M_{LL} = 712w_{LL}$	1990 K-ft	910 K-ft	1100 K-ft	3552 K-ft
	Shear: $V = 0.502P$[1]	18.1 K	1.21	21.9 K	$V_{DL} = 31.5w_{DL}$ $V_{LL} = 33.7w_{LL}$	94.5 K	43.1 K	52.2 K	168.6 K
	Axial Force: $F = 0.484P$[1]	17.4 K	1.21	21.1 K	$F_{DL} = 21.3w_{DL}$ $F_{LL} = 22.8w_{LL}$	67.0 K	29.2 K	35.4 K	123.5 K
Floorbeam (interior)	Assume identical to exterior, except for additional longitudinal force due to maximum difference in load of T_2 and T_2'.								
	$Shear_{y-y}$: $V_{y-y} = 0.29P$[1]	10.4 K	1.21	12.6 K	$V_{y-y} = 17.55w_{LL}$	--	22.5 K	27.2 K	39.8 K
	$Moment_{y-y}$: $M = 5.5P$	198 K-ft	1.21	240 K-ft	$M_{long} = 333w_{LL}$	--	326 K-ft	395 K-ft	635 K-ft

(1) P = 36 K at concentrated load for two lanes. (Assume acting at center of floorbeams.)
(2) Span = 110 ft for impact factor computation.
(3) w_{DL} = 3000 plf (est), w_{LL} = 1280 plf as two lanes of uniform load.
(4) Span = 220 ft for impact factor computation.

TABLE XIX. PRELIMINARY DESIGN: FRAME BRIDGE CONCEPT IN MODIFIED BRIDGE APPLICATION (SUPERSTRUCTURE MEMBERS)

Member	Design Value	Design Notes	Section	Area	Unit Weight	Length	Weight	Quantity	Total Weight
Principal Structural Members									
Tension									
$T_{1,3}$	227.5 K	Design as tension member, f_{all} = 20 ksi; A req'd = 11.35 sq in.	4-in. dia cable	12.6 sq in.	42.8 plf	42 ft	1.8 K	4	7.2 K
$T_{2,2}$	198.7 K	Design as tension member, f_{all} = 20 ksi, A req'd = 9.95 sq in.	3-3/4-in. cable	11.0 sq in.	37.4 plf	69 ft	2.6 K	4	10.4 K
Legs									
L		Design for moment.	Box girder: 84 × 42-in. flanges: 42 × 1-1/2-in; webs: 1/2 in.	207 sq in.	704 sq in.	8[illegible].4 ft	50.5 K	4	226.0 K
Axial Force	307.7 K								
Moment	8,157 K-ft								
Shear	114.3 K								
F:									
Axial Force	480.6 K	Design for moment	Box girder: 96 × 48-in. flanges: 48 × 1-1/2-in; webs: 9/16 in.	272 sq in.	925 plf	110 ft	92.5 K	1	92.5 K
Moment	12,450 K-ft								
Shear	11.1 K								
Floorbeam (exterior)									
Moment	3,552 K-ft	Plate girder design indicated S req'd = 2140 cu in.	48 × 18-in., 18 × 2-in. flanges, 5/16-in. web	106.0 sq in.	360 plf	61 ft	22.0 K	2	44.0 K
Shear	168.6 K								
Axial Force	123.5 K								
Floorbeam (interior)									
$Moment_{x-x}$	3,552 K-ft	Box girder design indicated.	54 × 36-in., 36 × 2-in. flanges, 3/8-in. web	182 sq in.	620 plf	61 ft	37.8 K	1	37.8 K
$Shear_{x-x}$	168.6 K								
Axial Force	123.5 K								
$Moment_{y-y}$	635 K-ft								
$Shear_{y-y}$	39.8 K								

Total Principal Structure	417.9 K
10% Misc.	41.8
Total Bridge Weight (excluding existing structure)	459.7 K

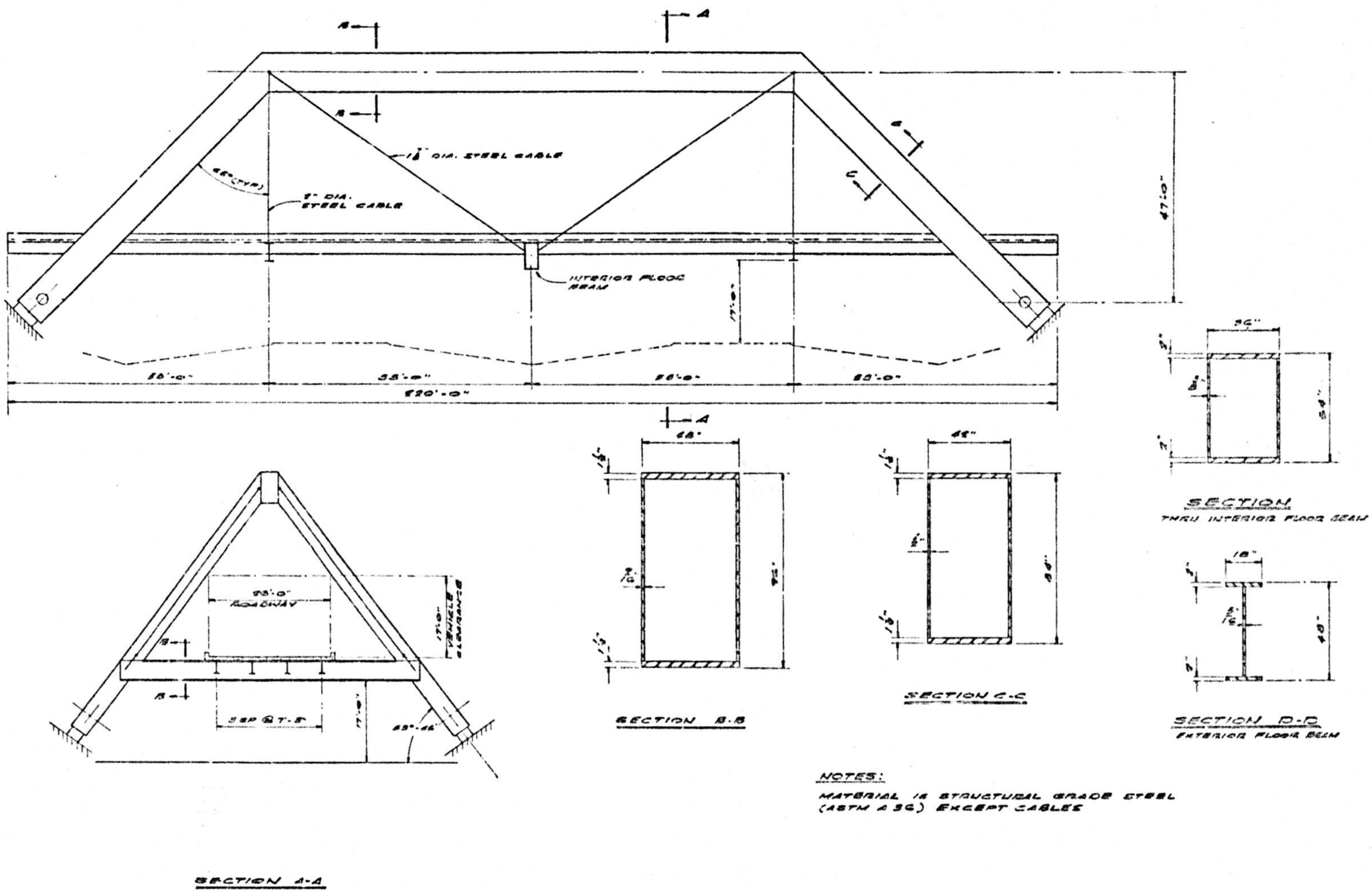
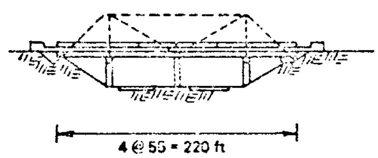


FIGURE 26. PRELIMINARY DESIGN DRAWING OF FRAME BRIDGE CONCEPT IN MODIFIED BRIDGE APPLICATION

TABLE XX. PRELIMINARY COST ESTIMATES: FRAME BRIDGE CONCEPT IN MODIFIED BRIDGE APPLICATION

Item(1)	Unit	Quantity(2)	Unit Cost(3)	Cost: Prototype Only	Cost: Quantity
1. Structural Steel, Girder	lb	81,600	\$0.25	\$ 20,400	
2. Structural Steel, Fab. (Frame)	lb	318,500	0.50(0.30)(4)	159,000	(95,500)
3. Steel Cable	lb	17,600	0.75(0.65)(4)	13,200	(11,400)
4. Misc. Steel	lb	41,800	0.30	12,600	
5. Remove Interior Bents (3 ea)	ls	-	-	6,000(5)	
6. Barricades, Warning System	ls	-	-	1,000(5)	
Total Estimated Cost (Bridge modification, excl. frame substructures)				\$212,200	(\$146,900)

(1) Stringer, deck and rail system salvaged intact from original bridge.
(2) Items 1 and 4 from Table XIX.
(3) Items 1 and 4 from first quarter (CY 69) Texas Highway Department construction cost data for in place units shown. These components are conventional bridge construction items.
(4) Items 2 and 3 are estimated in place unit costs for one-of-a-kind bridge installation. Procurement in quantity (unit and total costs) are shown in parentheses.
(5) Lump sum estimates based on modification of four span continuous bridge as illustrated below:



C. Concept Designs for Modifying Existing Bridges

Three additional bridge concepts* were identified in Section III, Paragraph C, as being potentially effective structural schemes for use in new bridge applications, although these schemes were not given the in-depth consideration received in preliminary design efforts. These concepts may also be considered in terms of their applicability to modifications of existing bridges. Three concept designs were developed (see Figures 14, 17, and 20) to provide the highway engineer with general appraisals of design and construction considerations identified with each of the concepts as employed in new bridge applications. In the paragraphs that follow, each of the three concepts is considered with respect to its applicability to modifying existing bridges to improve their safety from a horizontal clearance perspective.

1. Stayed Girder Bridge

The stayed girder bridge is uniquely configured to support a single, longitudinal box girder in the manner illustrated in Figure 12. For this reason, no easily adaptable configuration can be projected which could provide an effective modified bridge structural scheme.

2. Braced Arch Bridge

This single arch concept is aesthetically pleasing and, within the bounds of limitations previously expressed concerning new bridge construction, may possibly be employed in modifying existing bridges to allow removal of hazardous supporting structures. As noted previously, in the new bridge applications discussion, a significant limitation on employing this concept concerns out-of-plane loading of the single arch by live loads placed unsymmetrically widthwise on the bridge deck structure. The effects of such loadings on arch design have not been investigated to an extent which will allow final determination of feasibility; however, concept design calculations indicate that the bridging concept is potentially feasible.

The Braced Arch Bridge concept may be employed in modifying existing bridges to the extent that the concept is determined to be technically and economically feasible as a new bridge concept. The existing bridge types which may be modified include both simple and continuous spans over three or more supports. A general appraisal of the use of the Braced Arch Bridge concept in modifying existing bridges may be realized by reviewing the concept design information contained in Figures 15, 16, and 17, as well as in Tables XII and XIII.

*Stayed Girder Bridge, Braced Arch Bridge, and Leaning Piers Bridge.

3. Leaning Piers Bridge

The Leaning Piers Bridge concept may be employed in effecting modifications of existing bridges to accomplish removal of hazardous supporting structures. Limitations on the use of the concept, in general, as well as limitations on the use of the concept in the modified bridge application, raise questions concerning the effectiveness of the concept in the modified bridge application. Undesirable, but not unmanageable, aspects of the concept involve axial forces in the main horizontal girders. These forces can be accommodated in new bridge construction by employing continuous girders with appropriate restraints against buckling. Existing bridges to be modified using this concept must be two-span continuous structures capable of being braced to carry axial loads within continuous girders system. A general appraisal of the appearance and design characteristics of the Leaning Piers Concept employed to modify an existing bridge may be realized by reviewing the information and data contained in Figures 18, 19, and 20, and Tables XIV and XV.

V. PRELIMINARY DESIGNS FOR SIGN AND LIGHTING SYSTEM SUPPORT STRUCTURES

A structural design concept for supporting sign and lighting systems was selected for preliminary design attention from several schemes considered in the concept development and evaluation portion of the project.* This structural design concept is responsive to specific sign and lighting requirements (identified in the discussions which follow) rather than being advanced as a universally applicable configuration. Design criteria employed in developing preliminary designs include geometric, load, and aesthetic considerations. These criteria are presented and discussed, with the simplifying assumptions made, in Section II, Paragraph B.

A. Sign Support Structure Concept

New sign support concepts can be placed into a context compatible with definitions of sign-supporting structures contained in AASHO specifications. These definitions are shown in Figure 27 as follows: (1) balanced butterfly, (2) unbalanced butterfly, (3) cantilever, (4) sign bridge, (5) sign bridge cantilever, (6) structure mounted sign, and (7) road side sign. The two butterfly designs should not be used in new construction (and, where possible, they should be removed from existing construction) because their design application calls for placement in a "gore" area or in an area that must be in close proximity to a travel way. The sign bridge cantilever should find no future application for the same reason. Structure mounted signs are not within the scope of this study, although it is noted that use of bridge structures for signing purposes is probably not as prevalent as it should be. Roadside signs do not present hazardous massive support structures if they are positioned away from proximity to the travel way, or if they are configured to be of breakaway design. Thus, the remaining structures (i.e., the cantilever and the sign bridge) are the only types of sign supports requiring efforts directed toward the elimination of massive structures. The sign cantilever was selected for preliminary design attention because of its more frequent use, although several feasible sign cantilever and sign bridge concepts were advanced in the concept development portion of the program. Basic geometry of the sign cantilever concept selected for preliminary design is presented in Figure 28. Applications, analysis, preliminary design, and cost discussions are included for this cable supported structure in the following paragraphs.

1. Applications Discussion

A cantilevered sign support structure employing cable supports was selected for preliminary design consideration. This structural configuration may be employed at roadside and ramp locations which require

*Included in Volume I (Research Information).

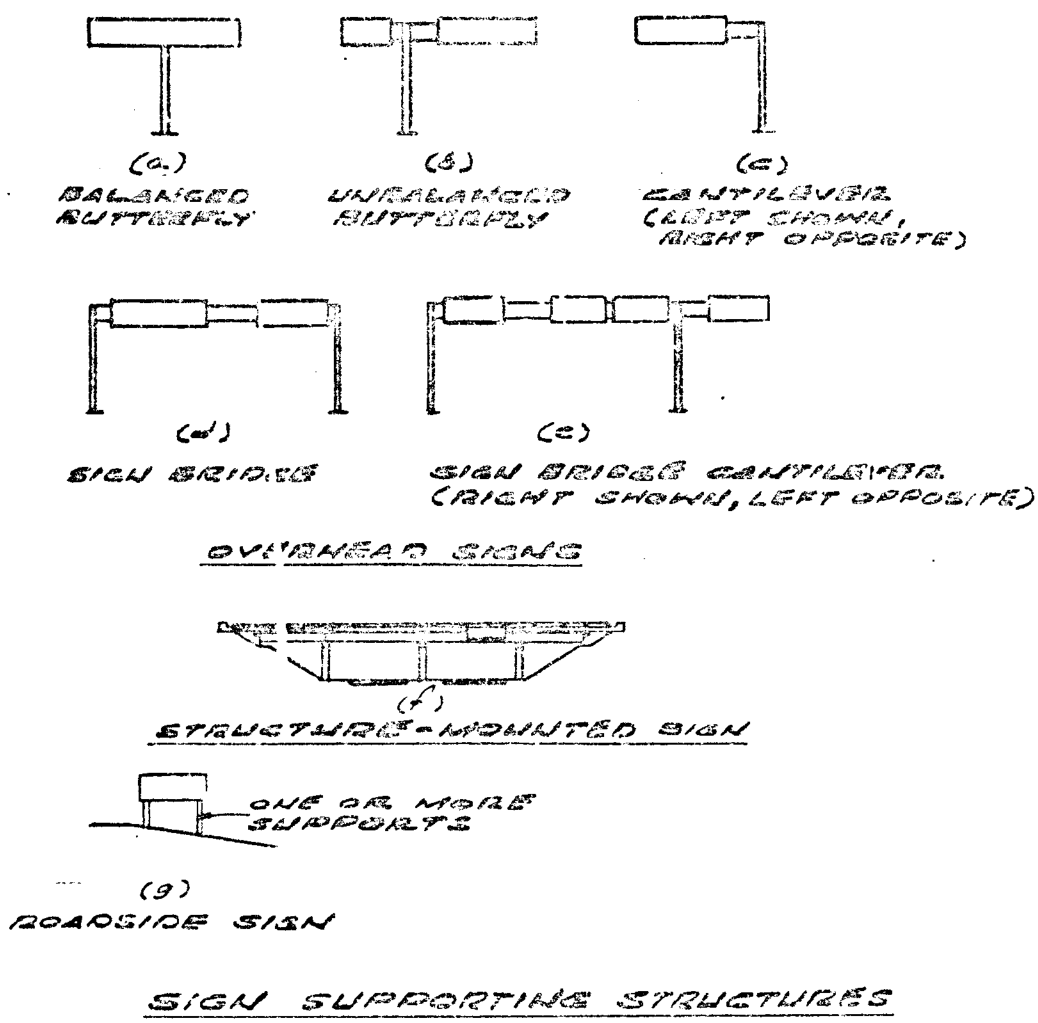


FIGURE 27. SIGN SUPPORTING STRUCTURES DEFINED IN AASHO SIGN SUPPORT SPECIFICATIONS(3)

information display over or adjacent to ramps or the outside traffic lane. Situations which require information displays over more than one traffic lane will require a sign bridge, or signs may be located on a bridge structure.

2. Analysis Discussion

The clearance requirements illustrated in Figure 1 for sign support structures are particularly demanding for cantilevered structural configurations. A free standing unit meeting stress and deflection requirements(3) would probably be curved in configuration and be relatively heavy in design. A cable supported structure such as the configuration presented in Figure 28 has potential for minimizing the weight of the system; however, there are disadvantages associated with this configuration also.

The structural system described schematically in Figure 28 consists of a compression member (main pole unit) inclined at 45°, two supporting cables which have been pretensioned to a specified force magnitude, and a cantilever arm which supports the sign. The geometry of the system places the sign above the minimum clearance requirement of 18 feet; this constitutes a major disadvantage. The system is statically indeterminate, therefore, it was necessary to develop a detailed method of analysis to guide the preliminary design; this method of analysis is presented in Appendix F. It is noted that the analysis method is based on a three step erection sequence: (1) Erect main pole unit and cantilever arm (unit will stand under its own weight), (2) Install cables and pretension to amount of force specified in Table XXI, or to a force magnitude which exactly compensates for the dead load deflection of the compression member at its upper end, and (3) Install sign unit. The cable preload specified will assure that the cables will remain in tension when the system is subjected to any combination of maximum design load conditions. This initial, pretensioned condition assures the integrity of the system, reduces deflection under load and minimizes adverse dynamic effects.

3. Design Discussion

A preliminary design for a cable supported, cantilever sign support structure is presented in Figure 29. The compression member (main pole unit) has been designed as a circular tube with a constant cross section. Torsional stresses in the main pole unit which result from wind loads on the face of the sign panel must be carried by this member. Axial forces, end moments and torsion must be considered in designing the main pole unit. The design is "balanced" in the initial (pretensioned) condition so that stresses in the system remain within limits under all load conditions, and that the cables are always in tension.

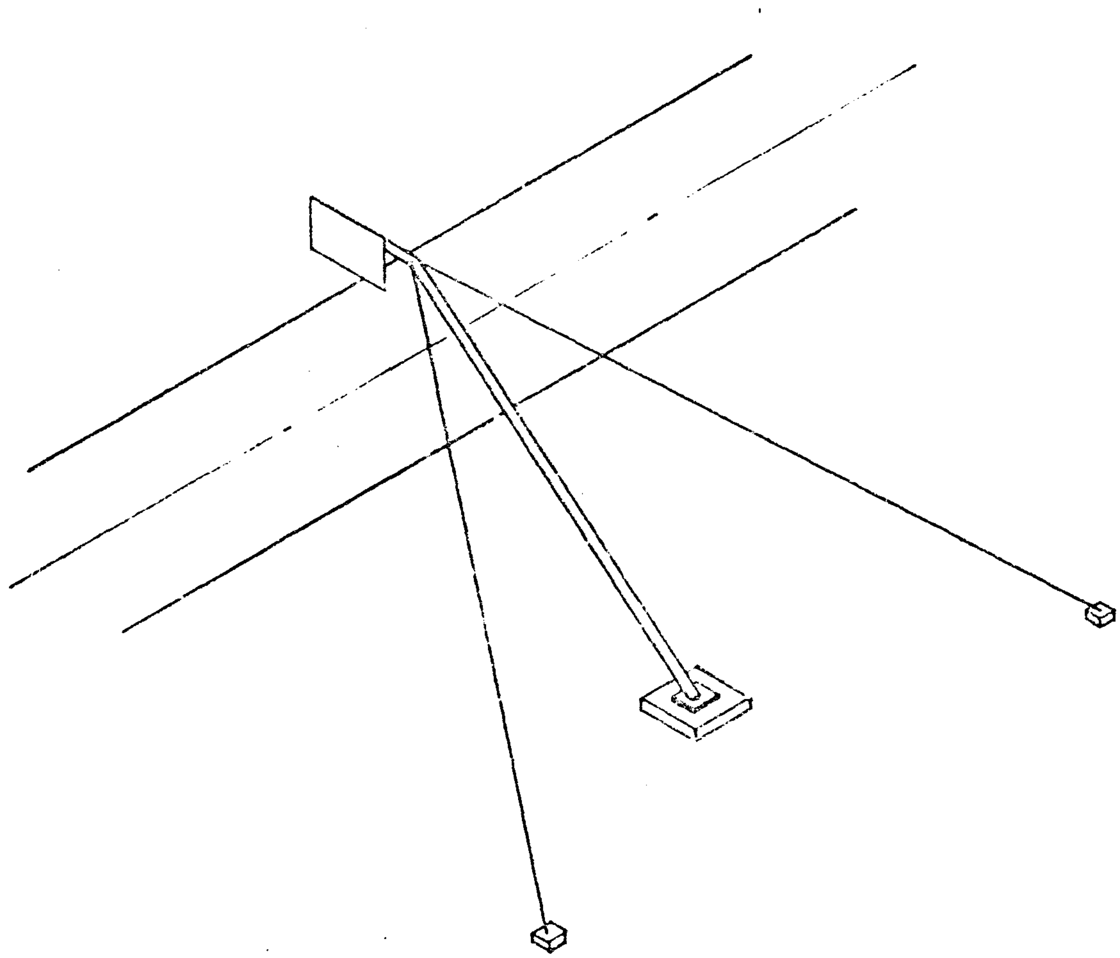


FIGURE 28. ISOMETRIC SCHEMATIC DRAWING OF SIGN CANTILEVER DESIGN CONCEPT

TABLE XXI. SUMMARY OF GEOMETRY, FORCES AND STRESSES: CABLE SUPPORTED SIGN STRUCTURE

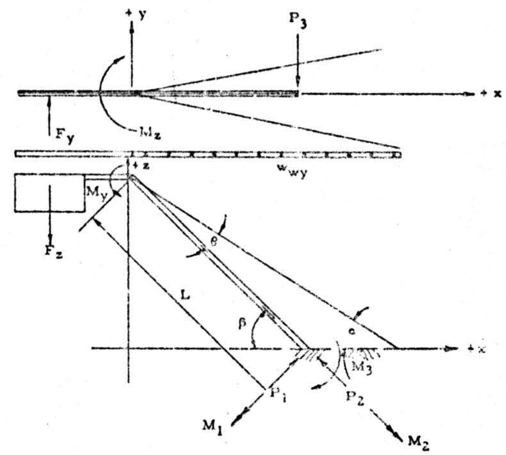
(a) Geometry

Length of Comp. Member L (ft)	L_x (ft)	L_z (ft)	Length of Cables ℓ_c (ft)	ℓ_{cx} (ft)	ℓ_{cy} (ft)	ℓ_{cz} (ft)	β (deg)	α (deg)	θ (deg)
47.4	33.5	33.5	72.7	51.5	38.75	33.5	45	33	12

(b) Summary of Applied Loads, Design Moments and Forces

Applied Load	P_1 (lb)	P_2 (lb)	P_3 (lb)	M_1 (ft-lb)	M_2 (ft-lb)	M_3 (ft-lb)	Cable Tension (lb)
F_z = 1000 lb	169	3820	--	--	--	8,000	1520
F_y = 3300 lb	--	--	136	6,420	--	--	±3100
w_{wy} = 50 psf	--	--	852	15,700	--	--	±800
$w^{(2)}_{D.L.}$ = 48.5 plf *	1620	1620	--	--	--	54,700	--
PRELOAD	-1070	2920	--	--	--	-50,700	2585
$M_y^{(1)}$ = 13,500 ft-lb	320	870	--	--	--	1,670	910
$M_z^{(1)}$ = 31,500 ft-lb	--	--	953	-13,750	22,200	--	±795
Total (max)	1039	9230	1941	8,370	22,200	54,700	9710
Total (net)	1039	9230	1941	8,370	22,200	13,670	5015 ± 4695

(1) These moments are due to F_z and F_y when transferring these loads to cable, compression member intersection.
(2) Dead weight of compression member.
(3) See Appendix F.



(Left-Hand Rule for Moments)

(c) Summary of Allowable Loads and Design Stresses

Load or Stress	Formula	Design Information	Allowable Loads and Design Stresses
P_{cr}	$P_{cr} = (u_{cr})^2 \frac{EI\beta}{L^2}$	u_{cr} = 2.5	P_{cr} = 161,000 lb
f_{b1}	$\frac{M_1 c}{I}$	M_{1max} = 9370 ft lb, c = 6.375 in., I = 279.3 in^4	f_{b1} = 2290 psi
f_s	$\frac{M_2 c}{J}$	M_{2max} = 22,200 ft lb, c = 6.37 in., J = 558.6 in^4	f_s = 3060 psi
f_{b3}	$\frac{M_3 c}{I}$	M_{3max} = 54,700, c = 6.37 in., I = 279.3 in^4	f_{b3} = 15,000 psi
f_a	$\frac{P_2}{A}$	P_{2max} = 9230, A = 14.6 in^2	f_a = 632 psi
f_T	$\frac{F}{A}$	Maximum cable tensile stress = f_T	f_T = 49,500 psi

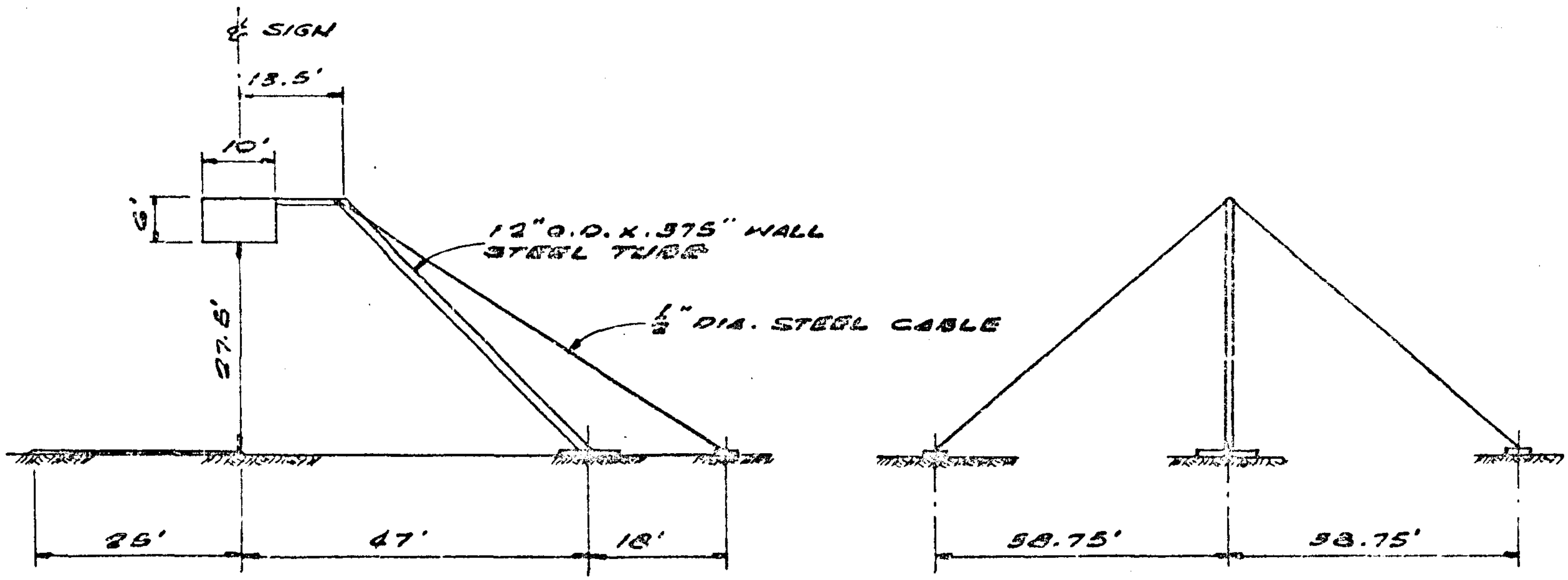


FIGURE 29. PRELIMINARY DESIGN FOR CABLE SUPPORTED
SIGN SUPPORT STRUCTURE

4. Cost Discussion

Costs of the preliminary design presented in Figure 29 may be considered only on a relative basis in comparison with a free-standing structure which meets identical geometric requirements. The cable supported system will weigh less than a comparable free-standing unit, since large bending moments in the free-standing structure which result from wind, ice and dead loads are minimized in the cable supported structure by the pretensioned cable supports. A free-standing unit which provides the same clearances would likely be curved in configuration, rather than straight, thus suggesting that such a unit may be more expensive to fabricate. Savings in weight and fabrication, in the case of the cable supported structure, may be offset by increased installation costs, although efficient erection procedures and equipment could be developed to minimize this expense.

B. Lighting System Support Structure Concept

Present-day lighting system support structures that properly position luminaires above the roadway can be grouped into two general categories: cantilever supported and overhead supported. The cantilever supported category includes discrete, free-standing structures such as the pole-arm unit. The overhead supported category includes bridge structures, as well as cable suspension systems. A specific cantilever supported scheme which may be employed to suspend luminaires at heights of 40, 50, and 60 feet above the roadway was selected for preliminary design consideration. This type of lighting system support structure was selected, in preference to the overhead supported structural type, because of its frequency of use in general lighting applications. Basic geometry of the cantilever supported lighting system concept selected for preliminary design is presented in Figure 30. Applications, analysis, preliminary design, and cost discussions are included for this cable supported structure in the following paragraphs.

1. Applications Discussion

A cable supported structure intended for general application along the edges of the roadway, including ramp and intersection sites, was selected for preliminary design from several cantilever and overhead concepts considered.* Several specific geometric configurations of the basic concept were considered and preliminary designs for each of three luminaire heights (40 feet, 50 feet, and 60 feet) were developed. A 30-foot luminaire height using the cable supported concept was determined to be impractical and not in line with trends toward higher liminaire positions. The three preliminary designs developed are similar in configuration. The compression member (main pole unit) must clear a point defined by the 18-foot vertical

*See Volume I (Research Information) for concept designs.

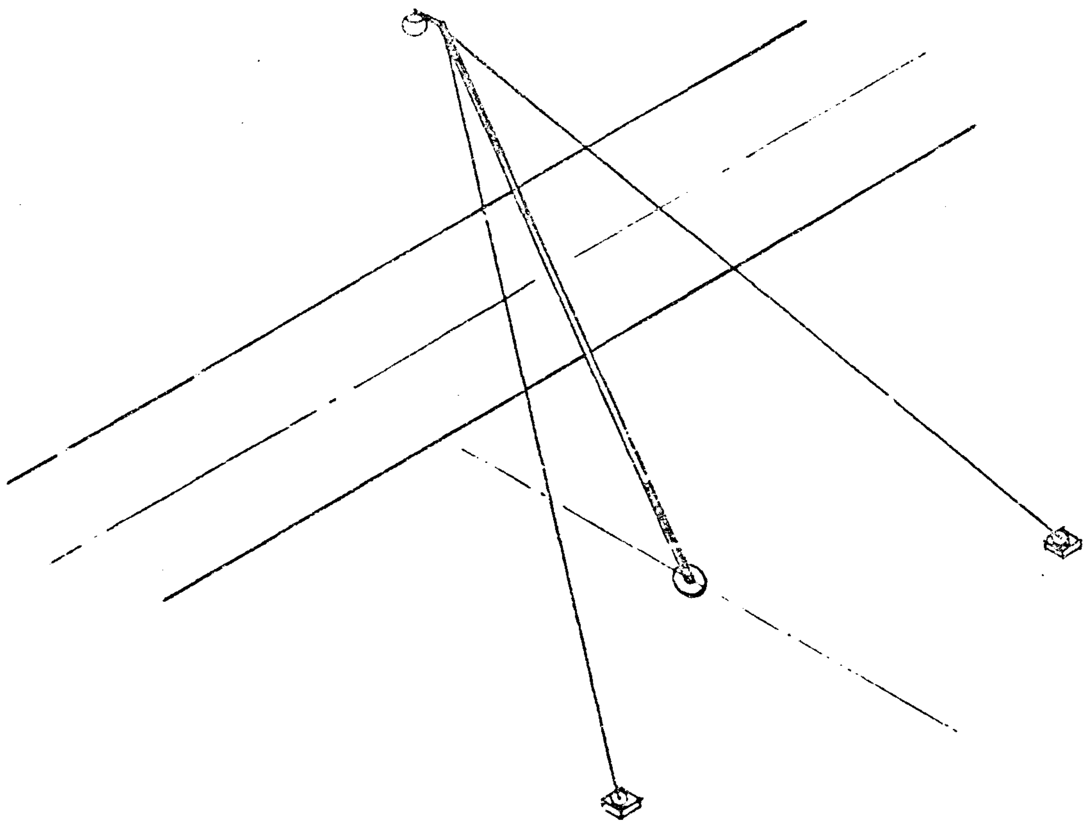


FIGURE 30. ISOMETRIC SCHEMATIC DRAWING OF CANTILEVER-SUPPORTED LIGHTING SYSTEM CONCEPT

and 30-foot horizontal clearance requirements prescribed by Figure 1. The luminaire itself is located as close as possible to the intersection of the cables and compression members to minimize induced bending moments at the pole end. In the configurations presented the luminaire is positioned over the outside edge of the roadway.

2. Analysis Discussion

The structural system described schematically in Figure 30 consists of a compression member (main pole unit) and two cables which are pretensioned to a specified force magnitude. The system is indeterminate; therefore, a detailed method of analysis was developed to guide the preliminary design. The method of analysis is presented in Appendix F. There are no codes or specifications currently in use which govern light standard structural design; therefore, conservative wind and ice loads were assumed for use in designing the three lighting systems. A wind force of 50 psf and an ice load of 3 psf were employed for the main pole unit and luminaire. The wind force was considered to act horizontally in any direction so as to produce maximum conditions for design. The erection and cable tensioning sequence is important and will influence analysis and design, therefore, the construction procedure must follow the following steps:

(1) Erect compression member, anchor to footing (main pole unit will stand under its own weight) and

(2) Install cables and pretension to specific amount of force specified in Table XXII or to a force magnitude which exactly compensates for the dead load deflection of compression member at its upper end.

Table XXII summarizes the forces which result from dead, wind, and ice loads, as well as the preload, for the three preliminary designs. Notice that the specified cable preload will assure that the cables will remain in tension when the system is subjected to any combination of maximum design load conditions. This initial pretensioned condition assures the integrity of the system and minimizes adverse dynamic effects.

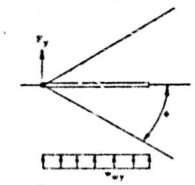
3. Design Discussion

The geometries of th three preliminary designs are compared in Figure 31. The compression members have been designed as circular tubes with constant cross sections. However, the nature of the loads imposed on the compression member indicate that a tapered cross section, with the maximum section at the lower, fixed end, would represent the optimum design. Use of a tapered section will complicate the indeterminate analysis, although the weight savings made possible through this effort may justify the analysis

TABLE XXII. SUMMARY OF GEOMETRY, LOADS, AND STRESSES FOR LIGHTING SUPPORT SYSTEMS

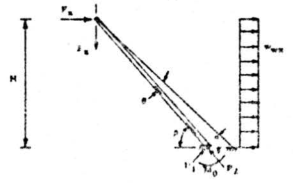
(a) Geometry

Luminaire Height, H (ft)	Compression Member Length, L (ft)	L_x (ft)	L_z (ft)	Cable Length ℓ_c (ft)	ℓ_{cx} (ft)	ℓ_{cy} (ft)	ℓ_{cz} (ft)	α (deg)	β (deg)	θ (deg)	ϕ (deg)
40	65.7	52.2	40	87.2	67	38.7	40	30.3	37.6	6.8	30
50	67.7	45.5	50	92.2	67	38.7	50	36.7	47.8	11.1	30
60	73.2	41.8	60	98.0	67	38.7	60	41.8	55.3	13.5	30



(b) Applied Loads(1)

Luminaire Height (ft)	F_x (ft)	F_y (lb)	F_z (lb)	w_{wx} (psf)	$w_{D.L.}$(2) (plf)
40	-200	0	180	-50	15.2
50	-200	0	180	-50	34.4
60	-200	0	180	-50	48.5



(c) Design Loads

Applied Load	40-Ft Luminaire Height				50-Ft Luminaire Height				60-Ft Luminaire Height			
	Cable Tension (lb)	P_1 (lb)	P_2 (lb)	M_o (ft-lb)	Cable Tension (lb)	P_1 (lb)	P_2 (lb)	M_o (ft-lb)	Cable Tension (lb)	P_1 (lb)	P_2 (lb)	M_o (ft-lb)
F_x	410	17	366	890	382	14		915	346	15	503	1,110
F_z	633	13	1230	1,100	314	11	699	760	218	10	535	700
w_{wx}	2710	1103	2630	17,700	1890	1397	1500	24,400	1310	2436	240	68,000
Preload	1440	-302	2570	-20,000	1660	-584	2970	-39,600	2200	-943	3930	-55,500
$w_{D.L.}$	--	798	610	26,400	4236	1560	1720	53,000	--	2020	2930	74,000
Total	5193	1629	7406	26,090	4236	2398	7436	39,475	4074	3538	8130	88,310

(d) Summary of Allowable Loads and Maximum Stresses

Luminaire Height, H (ft)	$\ell_{c_{xz}}/L \cos\theta$	$(u)_{cr}$(3)	$P_{cr} = (u)_{cr}\frac{EI}{L^2}$(4) (lb)	M_{max}(5) (ft-lb)	$S = \frac{I}{C}$ (in.^{3})	$f_b = \frac{12 M_{max}}{S}$(6) (psi)	A (in.^{2})	$f_a = \frac{P_2}{A}$(7) (psi)	$f_{max} = f_b + f_a$(8) (psi)
40	1.2	2.654	22,500	26,400	12.25	25,800	5.7	1300	26,100
50	1.26	2.55	47,500	53,000	29.9	21,200	11.9	625	21,825
60	1.27	2.55	70,500	88,310	43.8	20,200	14.0	530	20,750

(1) For maximum design condition.
(2) Dead load of compression member.
(3) See Table F.II in Appendix F.
(4) See Appendix F. Critical load for elastic stability while preload is being applied.
(5) M_{max} = Maximum bending moment for compression member.
(6) f_b = Bending stress due to maximum bending moment.
(7) f_a = Axial stress for maximum loading.
(8) f_{max} = Maximum stress for compression member. These stress levels are near design allowables presently being employed in bridge design.

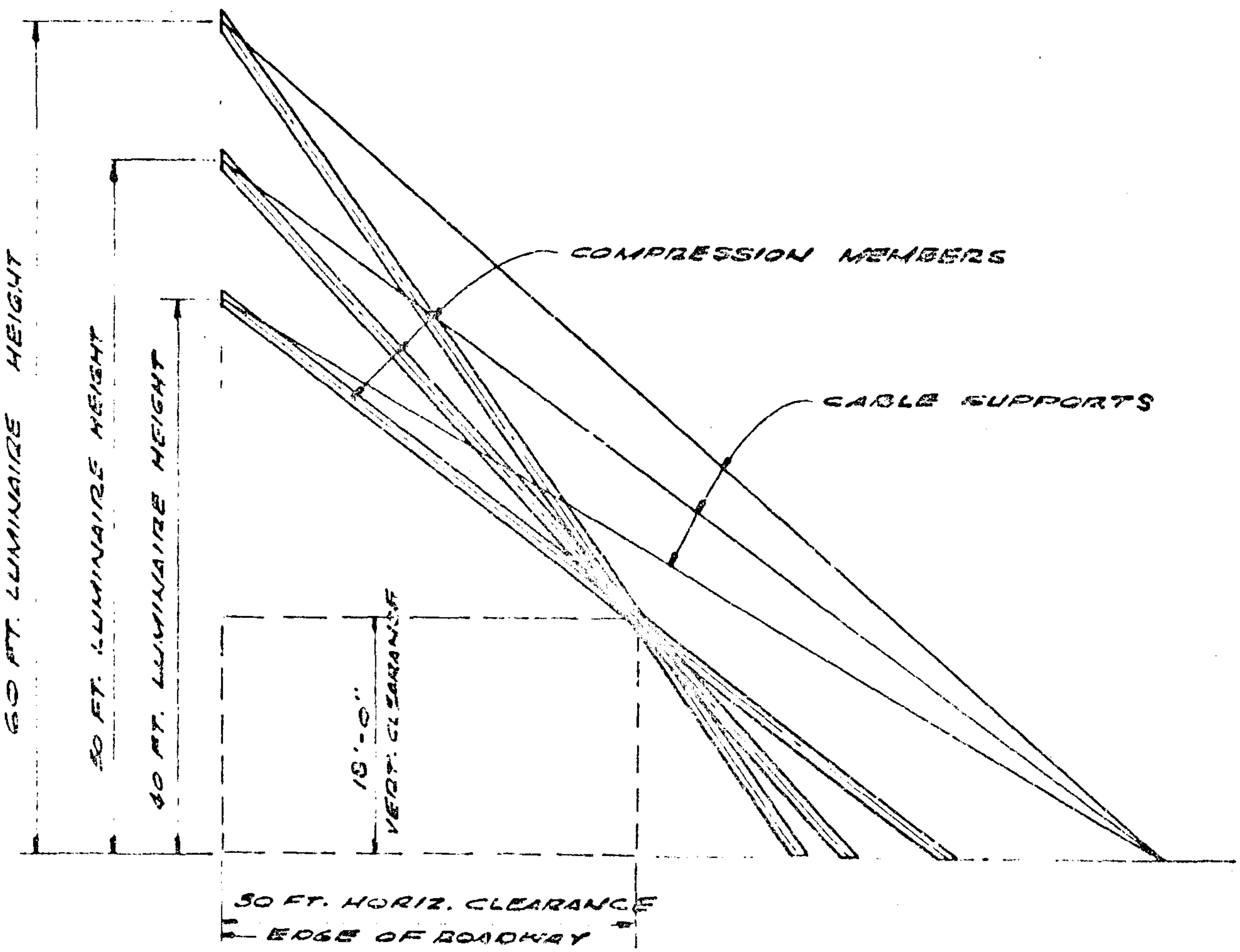


FIGURE 31. COMPARISON OF GEOMETRIES OF THREE LIGHTING SYSTEM PRELIMINARY DESIGNS

refinement if large quantities of the main pole units are to be produced. The compression member functions as a beam-column, therefore, axial forces and end moments must be considered in analyzing the unit's stability. Preliminary designs for the three luminaire heights are presented in Figure 32. The cable size indicated was determined by stiffness, rather than stress, considerations. In this regard, it is noted that the design is "balanced" in the initial (pretensioned) condition so that stresses in the system remain within limits under all load conditions, and that the cables are always in tension.

4. Cost Discussion

It would be difficult to determine exact costs of the lighting system designs presented in Figure 32. The system should weigh less than a free-standing unit which provides the same clearance, since large bending moments in the free-standing structure that result from wind, ice and dead loads are eliminated by the pretensioned cable supports. Savings in weight may be offset by increased installation costs, although it appears that efficient erection procedures and equipment could be easily developed. A free-standing unit which provides the same clearances would likely be curved in configuration, rather than straight, thus implying that such a unit may be more expensive to fabricate.

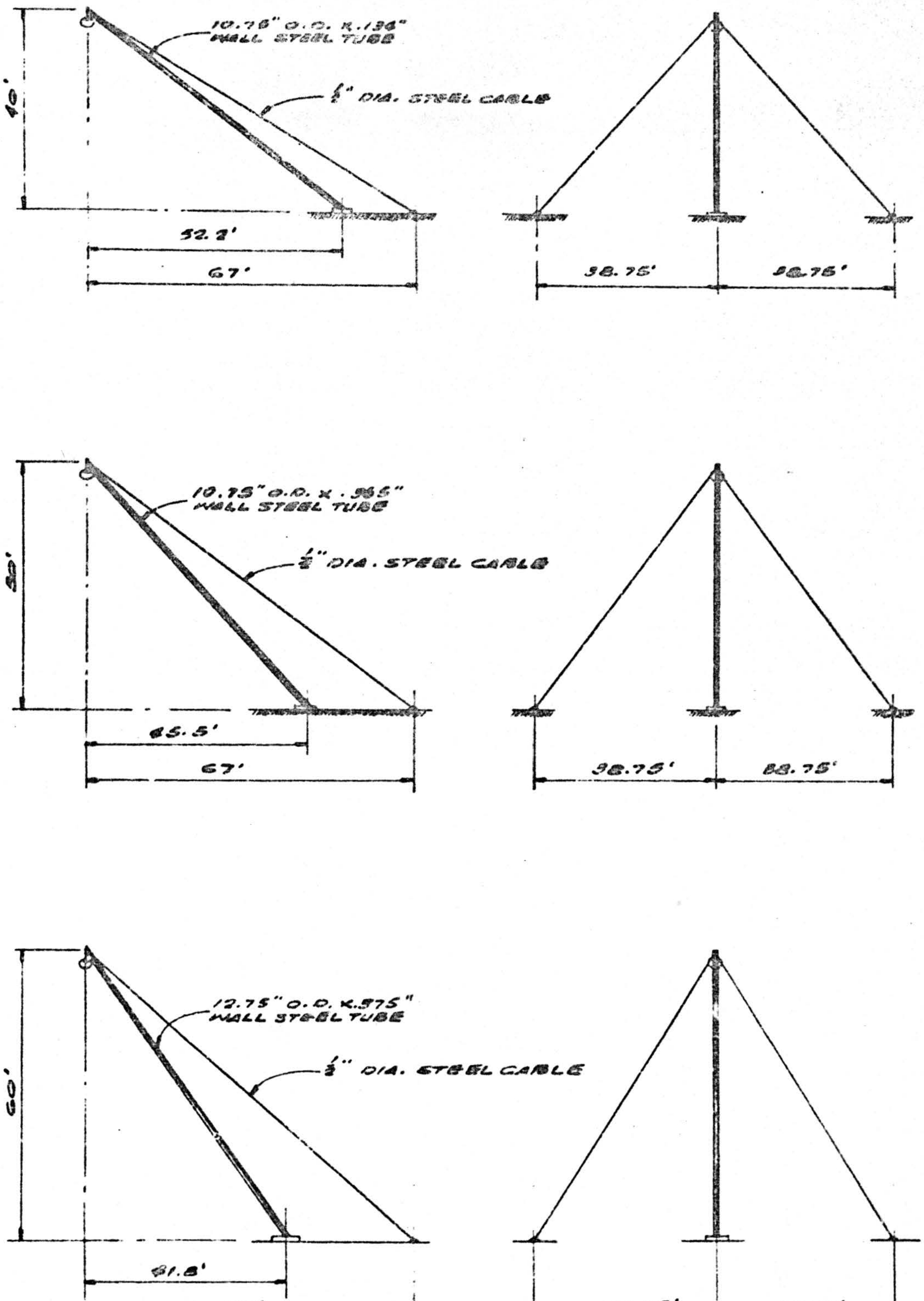


FIGURE 32. PRELIMINARY DESIGNS FOR CABLE SUPPORTED LIGHTING SYSTEM STRUCTURES (40-, 50- AND 60-FOOT LUMINAIRE HEIGHTS)

VI. SUMMARY AND CONCLUSIONS

The structural concepts and preliminary designs for highway appurtenances presented in this volume of the technical report represent the results of research efforts, comparative evaluations, and preliminary design activities conducted as a part of the Bureau of Public Roads' 4S Program. Preliminary designs presented for new bridges, modified existing bridges, and sign and light support structures were developed in response to geometric criteria suggested by recent trends in requirements for highway safety. In order to include as much quantitative information as possible in presenting preliminary designs, specific structures were defined for analysis and design. The resulting preliminary designs are not universally applicable; however, the quantitative information presented will provide the highway engineer with data for making appraisals of the structural effectiveness, economics, design, and construction complexities, and aesthetics regarding the various conceptual structures.

Specific observations related to individual structures concepts are presented in the portions of the report concerned with presentations of preliminary designs. In addition to these specific observations, the common geometric requirements upon which the preliminary designs were based permits additional, more general observations concerning the relative merits of the several concepts within each of the two new structures concepts categories [i. e., (1) bridges and (2) sign and lighting system support structures]. These general, comparative types of observations are summarized in the following paragraphs.

A. Bridge Concepts Evaluations

In comparing the three cable-supported bridge concepts presented as preliminary designs in Sections III and IV, the number of factors that may be considered are relatively few in number, when speaking in general terms. Economics is an important comparison parameter; in fact, it is probably the most accurate measure of concept effectiveness. Other important comparison considerations are design flexibility, aesthetics, site adaptability, and relative safety. The last factor is included because the concepts presented may appear to possess potentials for catastrophic failure in accident situations which result in the loss of one or more of the cable supports. Each of the comparison parameters identified above will be employed in the following discussions to effect a relative ranking of the bridge concepts within their respective categories as a means of summarizing and presenting evaluations and conclusions.

1. Economics

Preliminary cost estimates were prepared for the specific preliminary designs presented. The estimates were based on current unit prices, estimates of the costs of unconventional construction, and limited experience with bridge modifications. While the cost data are quantitative, they should be considered to provide only approximate projections of actual costs. In comparing the estimated costs of the two Bridle Bridge concept configurations with the estimated cost of the Leaning Arches Bridge concept, no clear ranking based on economics is discernible. Although the Bridle Bridge-Hinged Configuration has the lower projected costs, the estimates are sufficiently similar to indicate that there is no substantial difference between the costs of these two concpets. It may be conjectured, however, that had the geometric requirements been such that a longer span was required, the Leaning Arches Bridge could have proven to be the most economical.

In considering the estimated costs of preliminary designs identified with modifications of existing structures, the Leaning Arches Bridge concept is clearly the more economical, when compared to the Frame Bridge concept. Although it was pointed out in the preliminary design discussion that the Frame Bridge could experience significant weight reduction through design optimization, it is not likely that this savings would make it competitive with the Leaning Arches Bridge employed in a similar modification application. The cost estimates reflect an economic advantage in salvaging the existing structure, as opposed to constructing a completely new bridge, when employing the Leaning Arches concept. If the existing bridge were replaced, rather than modified, cost of removing the existing spans would have to be considered in conjunction with new bridge costs reflected in Table III. Additional bracing which must be added to the existing spans to resist increased lateral flexibility may, in some cases, rule out the use of existing spans; however, specific bridge situations would need to be considered on an individual basis to provide a complete picture of the complexities involved in retaining the existing spans.

2. Design Flexibility

In considering potential uses of the concepts presented, the highway engineer will be confronted with a multitude of geometric situations in addition to the specific 0° skew situation (Figure 1) used for developing preliminary designs in this study. He may find certain of these design concepts to be adaptable over a wide range of situations, while others will be limited by the clear span capabilities of the concept. The Leaning Arches Bridge appears to be the more efficient structural concept when spans longer than the specific example span are considered. The Bridle Bridge could conceivably utilize additional cables to provide more than one intermediate support for the spanning girder, although this, in effect, constitutes a different design. In this regard, it must be emphasized that the preliminary designs illustrated serve only to introduce the structural concept and design methodology; they are not advanced as the answer to all situations. As the designer becomes familiar with the feasible concepts, design refinements which may provide more desirable solutions will become evident. It appears that the Leaning Arches Bridge provides the designer with considerable flexibility in being able to accommodate a range of bridging requirements.

3. Aesthetics

The parameter identified as aesthetics provides the basis for effecting contrasts between concepts in accomplishing a general type of comparative evaluation. In order of their more pleasing appearance, the three feasible bridge concepts may be ranked: (1) Leaning Arches Bridge, (2) Bridle Bridge, and (3) Frame Bridge. The Leaning Arches Bridge ranks first in appearance, although the Bridle Bridge is not aesthetically unpleasing. The Frame Bridge is not considered to possess aesthetic appeal.

4. Site Adaptability

Certain sites may limit the designer in the available number of applicable bridge concept alternatives. The adaptability of the Leaning Arches Bridge to roadway cut locations has been discussed. This type of site location is ideal when considering foundation requirements, as well as when considering aesthetics. The Bridle Bridge span does not have overhead obstructions over a large portion of its span and might be effectively used where clearances are critical above the overpass (e. g., in multilevel interchanges). However, cable ties from the tower which terminate off of the structure could present an obstacle to general application of the Bridle Bridge concept (e. g., in areas where a frontage road intersects the crossing roadway near the structure). Vertical clearance requirements might eliminate crossing of the access roadway with the cable ties. The Frame Bridge and the Leaning Arches Bridge are comparable from a site adaptability standpoint.

5. Safety

A definite disadvantage of the cable-supported concepts presented in this study concerns the possibility of catastrophic failures of the structures if loss of one or more cable supports is experiencd. The concepts which are adaptable to simple spans configurations and the concepts which employ hinges in a span are particularly vulnerable to catastrophic failure resulting from loss of cable support. Treatment of this subject in new bridge design codes for cable-supported structures will be necessary if these types of bridges become extensively used. There are two courses of action which can be pursued to respond to this problem. The first would be to require the bridge design to be such that loss of a single cable support would not result in failure (i. e., design the bridge to support dead and live loads with a critical cable support removed). The second course of action would be to provide the cables with adequate protection from vehicles impacting the bridge rail, through provision of a rigid bridge railing which limits deflections from a specified impact to values less than the clearance provided between the rail and cable. A thorough study of this problem has not been conducted; however, the ranking for the new structures considered, with regard to this type of safety, is: (1) Leaning Arches Bridge, (2) Bridle Bridge (continuous), and (3) Bridle Bridge (hinged). For existing structure modification concepts, the two schemes presented (Leaning Arches and Frame Bridges) are essentially equal with regard to relative safety, measured in terms of effects of loss of cable supports.

6. Bridge Evaluation Summary

Bridge concept evaluations are summarized in Table XXIII. Although it would be necessary to attach weighted values to the rankings introduced in previous paragraphs and summarized in Table XXIII in order to establish overall rankings, it appears that the Leaning Arches Bridge is the most effective overall concept within the new structures category. For the modification of existing structures category, it is evident that the Leaning Arches Bridge is clearly the most effective overall.

Other bridge concepts which were not carried to the preliminary design stage, but which showed promise as being feasible, are the Stayed Girder, Braced Arch, and Leaning Piers Bridges. For new structures applications, the Stayed Girder is apparently the most effective of the three in responding to economic, design flexibility, aesthetic, and safety requirements. Since this concept is not applicable to applications concerned with the modification of existing structures, the Braced Arch Bridge concept provides the more desirable situation, when compared to the Leaning Piers concept, in responding to modified existing bridge requirements.

B. Sign and Lighting System Support Structure Concepts Evaluations

The cable supported sign and lighting system support structures preliminary designs presented in Figures 29 and 32 concern single sign or luminaire units which are commonly employed along the roadway and at ramp locations. Both systems employ the same design concept, i.e., a pretensioned cable supported configuration with threefold design objectives: stabilizing the structure against adverse dynamic effects, minimizing deflections, and saving total system weight. The pretensioned initial condition assures that no cable component will go "slack" under maximum adverse loadings and, thus, confirms the stability of the system.

1. Sign Support Structures

The cable supported sign support structure is perhaps the most simple, feasible solution to the relatively demanding geometric design

TABLE XXIII. BRIDGE CONCEPT EVALUATION SUMMARY

	Evaluation Parameters*				
Bridge Concept	Economics	Design Flexibility	Aesthetics	Site Adaptability	Safety
1. New Bridge Applications Category					
Leaning Arches Bridge	3	1	1	1	1
Bridle Bridge (Continuous Girder)	2	3	2	3	2
Bridle Bridge (Hinged Girder)	1	2	3	2	3
2. Modification of Existing Bridge Applications Category					
Leaning Arches Bridge	1	1	1	-†	-
Frame Bridge	2	2	2	-	-

*Rank within applications category, with respect to other concepts within the category.
†Equivalent rankings.

criteria of Figure 1. Although the system presented is responsive to design criteria and code requirements[3], the location of the sign panel itself above the minimum required clearance is a definite disadvantage. This position for the sign panel cannot be made lower, while retaining required clearances, without further sloping and lengthening the main pole unit. Angles with the horizontal less than the 45° angle illustrated in Figure 29 are not considered feasible. Similarly, curved pole units under pre-tension loads are not considered feasible. The alternate design, a free-standing single pole unit which places the sign panel 18 feet above the roadway, appears equally infeasible because of relatively large bending moments and deflections, as well as dynamic stability problems, which would result from wind loads acting on a sign panel mounted on a relatively long cantilever.

2. Lighting System Support Structure

The cable-supported lighting system support structures preliminary designs (Fig. 32) employ the same basic design concept as the sign support structure. These preliminary designs appear to be effective solutions to the demanding safety related geometric criteria of Figure 1. The luminaire heights of 40, 50 and 60 feet allow the main pole unit to assure reasonable angles of inclination, and the relatively small loads on the luminaire itself (when compared to the sign panel loads) minimize adverse effects of moments and torsion at the main pole unit's upper end. The "balanced" pre-tensioned structure is structurally efficient in responding to the loading conditions presented and should compare favorably with an alternate, free-standing cantilever design.

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