"STUDY OF A PROPOSED PRECAST-PRESTRESSED COMPOSITE BRIDGE SYSTEM";

FINAL REPORT.

MISSOURI STATE HIGHWAY DEPARTMENT
UNIVERSITY OF MISSOURI, COLUMBIA
BUREAU OF PUBLIC ROADS
List of Previous Reports on this Project


"STUDY OF A PROPOSED PRECAST-PRESTRESSED COMPOSITE BRIDGE SYSTEM"

Prepared for
MISSOURI STATE HIGHWAY DEPARTMENT

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MAY 1970

in cooperation with
U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
BUREAU OF PUBLIC ROADS

The opinions, findings, and conclusions expressed in this publication are not necessarily those of the Bureau of Public Roads.
Abstract

This final project report summarizes the research efforts on Missouri Cooperative Research Project No. 67-1, "Study of Precast-Prestressed Composite Slabs". A particular type of precast-prestressed composite box bridge deck system is proposed for use in highway bridge construction on primary and secondary roadways.

The basic concept and design procedure, as well as an evaluation of the structural performance and an economic evaluation of the proposed system, are summarized. In addition to these summaries, design recommendations for the proposed systems are made.
ACKNOWLEDGMENTS

The testing program reported herein was conducted in the Civil Engineering Laboratories of the University of Missouri, Columbia, Missouri. This work represents the final report on a study of a proposed precast-prestressed composite bridge system undertaken by the Engineering Experiment Station and sponsored by the Missouri State Highway Commission in cooperation with the Bureau of Public Roads of the Federal Highway Administration of the U. S. Department of Transportation.

Sincere appreciation is expressed to those organizations from whom cost information was obtained along with invaluable suggestions concerning the modification of the channel cross sections and processing of the cost information. These organizations include Nebraska Prestressed Concrete Company, Prestressed Concrete of Iowa, Inc., Tobin Construction Company, Wilkerson Construction Company, and Wilson Prestressed Concrete Company.

This work summarizes the four research phases of this project during the period from July 1, 1966 to January 31, 1969. All work performed on the project was under the supervision of J. R. Salmons. Messrs. J. R. Poepsel, S. Mokhtari, and W. J. Kagay were graduate research assistants working on the project.
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1.1 INTRODUCTION

With the adoption of the National Interstate Highway program and with additional emphasis on improved and more extensive state highway systems, bridge construction is of ever increasing importance. As a result of the expanded bridge construction program and the rapidly increasing construction costs, there is a great demand for more efficient and more economical bridge structures. Prefabricated and precast structures have exhibited these characteristics for the building industry as a whole while in some sections of the country precast bridge construction has lagged.

The bridge system proposed in this study is aimed at helping to overcome this deficiency by better utilizing the advantageous characteristics of precast concrete. The proposed system not only utilizes precasting but incorporates pre-tension prestressing and composite construction, as well as virtually eliminating forming of the bridge superstructure. Each of these features has independently demonstrated efficiency or economy and no less would be anticipated from the combination.

1.2 PROPOSED COMPOSITE-BOX BRIDGE SYSTEM

To make use of the advantages of precast prestressed composite construction a particular structural system is proposed. This system consists of a series of precast prestressed concrete channels with an interior void used in conjunction with a monolithic top slab of cast-in-place concrete. A typical section of the resulting "Composite-Box"
bridge deck is shown in Figure 1.1. The void in the bridge deck does not necessarily extend continuously through the entire span length but may be interrupted prior to reaching a pier and at intermediate points between piers. This provides solid bearing ends at the supports and furnishes a region for diaphragms to provide lateral continuity. An illustration of the general configuration of the proposed system at an interior bent is shown in Figure 1.2.

The interior void form can be fabricated from various materials, since the only strength requirement of the void is to support the load of the cast-in-place concrete during placing and the weight of the workmen during casting of the top slab. Since the configuration of this void form is such that it functions as a two-hinged arch, the form is stronger than a comparable flat void form. Corrugated metal, lightweight concrete, plywood, and plastic are but a few of the materials which could be used to fabricate the void form.

In addition to fulfilling the requirements of economy and service-ability, today's structure must also be esthetically pleasing. The proposed composite-box system would provide a smooth or textured, uncluttered appearance when viewed from beneath (as in the case of a highway overpass). Also, when viewed from the side, the structure would have a graceful appearance.

1.3 SCOPE OF STUDY

A particular type of precast-prestressed composite box bridge deck system is proposed for use in highway bridge construction on primary
Fig. 1.1 A Typical Cross-Section of the "Composite Box" Bridge Deck
Figure 1.2 Side View of Configuration at Support
and secondary roadways. In order to properly evaluate the system, the efforts were categorized into the following four interrelated phases, and the results are presented in interim reports:

1. "INVESTIGATION OF A PRESTRESSED-PRECAST BRIDGE SYSTEM" (1)*

The objects of the first phase of the study were to develop a design and analysis procedure for the proposed members and to investigate the characteristics of these members through a limited test program. In addition to the design procedure, this phase includes the fabrication of three 36-foot long composite beams designed to carry HS20-44 loading and presents results of flexural tests performed on these beams.

A computer program for the design of the proposed members and a design example are presented in Appendices of the report.

2. "MODEL STUDY OF THE PROPOSED PRECAST PRESTRESSED BRIDGE SYSTEM" (2)

The objects of the second phase of the study were to design a one-half scale concrete model of the member and to investigate the similitude relationship between the model and the prototype through a model test program. In addition to the theoretical approach, the study includes the fabrication of three 18-foot long composite members and five pre-stressed channels to be used for further bridge study.

Also, results of flexural tests performed on the three model beams are compared with the prototype test results and similitude correlations are considered.

* Numbers in parentheses refer to entries in bibliography.
3. "STUDY OF A PRECAST-PRESTRESSED MODEL BRIDGE SLAB" (3)

The object of the study in the third phase was to investigate the structural performance of the composite-box bridge system through tests carried out on a one-half scale concrete model simulating a 36-foot, two-lane, highway bridge span. Included in this report are the theoretical load distribution analysis, the experimental load transfer characteristics of the bridge slab under the action of various concentrated loads, and the comparison between the two sets of data.

In addition, ultimate load capacity and the mode of failure of the model bridge span, under the application of simulated wheel loads, are included.

4. "ECONOMIC EVALUATION OF THE PROPOSED PRECAST-PRESTRESSED BRIDGE SYSTEM" (4)

Because of the structural desirability and the advantages in construction of the proposed system, a cost analysis of this system was made. The purpose of the fourth phase of the study was, therefore, to determine the economic feasibility of constructing the superstructure of short-span highway bridges using the proposed system of precast-prestressed composite construction.

In determining the economic feasibility of the proposed system the most practical method was to compare the new system with contemporary bridges designed and constructed, or under construction, in Missouri.
In order to make realistic comparisons, the scope of this work was limited to the comparison of the cost of three specific highway bridges with the cost of the same bridges using the proposed system for the superstructures. The design of the superstructure for one of the bridges considered is included in Appendix B of this interim report. Design procedures were also developed and the modified computer program, together with a flow chart and sample output is presented in Appendix A of the interim report.
CHAPTER II
STRUCTURAL PERFORMANCE OF THE PROPOSED BRIDGE DECK SYSTEM

2.1 INTRODUCTION

The general structural performance of the proposed bridge deck system was evaluated through a series of full-scale and one-half scale model tests. The first two of these test series were conducted on single units, while the third test program utilized a five-unit monolithic section as shown in Figure 2.1. The total test program was designed to evaluate the following aspects of the structural performance of the system:

1. A check of the design procedure
2. A preliminary check of the method of construction
3. Behavior of the member under working loads and high overload conditions
4. Verification of the composite action between components of the members
5. The lateral load distribution characteristics of the members making up the multi-unit deck system
6. A check of the ultimate load characteristics of the proposed system

Whenever applicable, analytical predictions were used as a basis for evaluating the structural performance of the system.
Fig. 2.1 Test Specimens Used in the Study
2.2 INITIAL DESIGN OF THE BRIDGE SYSTEM

Prior to the detailed design of any member in the proposed system, it was desirable from a fabrication point of view for certain basic dimensions of the precast channels to be fixed. Since the moment capability of a prestressed member is primarily dependent upon the depth of the section and the location of prestressing force, the dimensions, with the exception of the depth, were fixed as those of the full-scale specimen shown in Figure 2.1. Although some slight modifications to the initial section based on recommendations from prestressed concrete producers were made during the economic evaluation study, the design and construction of the test sections were in accordance with those dimensions shown in Figure 2.1.

The simple support condition was initially used for both the dead and live load design. Again this condition was modified to account for continuity over intermediate supports for the live-load design during the economic evaluation phase of the study. Since all the tests were of simple span only, the continuity condition was not considered in design of the specimens.

The loading used for design of the laboratory specimens was a simulation of AASHO HS20-44. Due to the particular span length used in the full-scale test series, the trailer wheel locations were very nearly that of one-third point loading. Symmetrical loading is desirable for testing and, as a result, the specimens were tested using the one-third point loading condition. The error involved in this simulation was examined and found to be small (3). The initial and final design procedures developed for the member will accept the normal range of
AASHO standard loadings.

The actual detailed design of the test specimens was made, with one exception, in accordance with AASHO specifications with the assumption of complete composite action and the loading and support conditions previously stated. The exception consisted of reducing the top slab reinforcing to approximately 50% of the AASHO requirement for the single units since the method of loading was such that the load was distributed over the entire width of the member. A detailed example of the preliminary and final design procedures as well as a computer program for the design was presented in the Appendix of Interim Report No. 68-8. The revised design procedure and program were presented in Interim Report No. 69-1. An analysis of members designed by this procedure was used as a basis for comparison of the test results with predicted behavior.

2.3 COMPOSITE BEHAVIOR OF THE MEMBERS

Since one of the prime concerns of the structural performance of the proposed system was the composite action between the components of the member, portions of each test series were designed to evaluate load-strain, load-slip, and load-slab separation relationships. Each of these quantities independently indicates the degree of composite action between components of the deck system.

Load-Strain Relationships:

The members used in all of the test series were instrumented with six-inch strain meters at a minimum of four locations over the depth of the member. Generally the sets of strain meters were located at mid-
span, however, some measurements were made at the mid-point of the shear span. The strain distributions in the shear span for the single-member tests indicate a slight discontinuity at the interface of the two components prior to failure which corresponded to the actual failure mode. On the other hand, the strain distributions resulting from the test of the model bridge slab indicate complete composite action prior to failure which again corresponds to the actual mode of failure.

Figures 2.2, 2.3 and 2.4 show typical strain distributions at various load levels for each of the three test series respectively as described in Figure 2.1. Even though failure of the single members was premature due to loss of composite action, large discontinuities of strain were not evident prior to failure.

The similarity of strain distributions between that of the prototype and model members can be noted by comparing Figures 2.2 and 2.3. This comparison was more completely examined in Interim Report No. 68-9. These results show that the strains of the prototype can be predicted from those of the models with a reasonable degree of accuracy. The predictability constitutes part of the similitude relationships used to extend the one-half scale bridge slab test results to a full-scale bridge slab.

Load-Slip, Load-Slab Separation Relationships:

Each member of the test series was instrumented with dial indicators at four to six locations along the length of the member to measure slip and separation between the two components of the composite
Fig. 2.2 Strain Distribution at Midspan for Test Series No. 1
Fig. 2.3 Strain Distribution at Midspan for Test Series No. 2
Fig. 2.4 Strain Distribution in Exterior Unit for Test Series No. 3
member. In general, the data obtained from each of the test series were of very little value. Some small discontinuities in strains were observed in the shear span during the tests of the full-scale single members; however, the magnitude of these discontinuities was not sufficient to produce meaningful results. The results from the one-half scale bridge slab test indicated no discernible slips or slab separations which again complies with the load displacement results and the mode of failure of the slab.

2.4 LOAD-DISPLACEMENT RELATIONSHIPS

Dead-Load Displacement Relationships:

A displacement history was maintained for all of the channels used in each of the tests series. The displacement records were made for the members from the time of release of the prestressing strand through placing of the composite slab up to the time of application of the live load. A detailed description of the method of obtaining the displacements was presented in Interim Report 68-8 and 68-9. These measured values were compared with those predicted by conventional methods assuming a creep coefficient of 1.50 and with creep occurring over a 30 day period. In general, these predicted and measured values compared well for both the full-scale and the one-half scale members. A typical set of these values is shown in Figure 2.5.

Live Load-Displacement Relationships:

The live load-displacement relationships were evaluated both analytically and experimentally for each member in each test series.
Fig. 2.5 Typical Channel Deflection vs. Time
The load displacement relationships were predicted based on an idealized section and the assumption of complete composite action between the two components of the members. The idealization considered an equivalent box section and accounted for the variation in concrete strength between the channels and the cast-in-place deck through both the elastic and inelastic ranges. The deflection predictions were made by the conventional elastic theory through that portion of the loading. The inelastic portion of the predicted deflection curve was obtained by considering the curvature of the member after the section was cracked and after the concrete stresses were no longer proportionate to the concrete strain. Both the stress-strain relationships for the concrete and prestressing steel were idealized for these predictions. The predictions were made by an iteration process at discrete values of concrete strain. Through the use of equilibrium relations corresponding points on the load-displacement were determined. Straight line segments connecting these points produce an approximation for the theoretical load-displacement relationship.

Typical comparisons of the predicted and measured live load-midspan displacement are shown in Figure 2.6. It can be noted that there is very good correlation between the experimental and the predicted values even though several approximations were involved in the prediction method. Again it is evident from these comparisons that virtually complete composite action took place up to the failure load. In addition, for the case of the one-half scale bridge deck the ultimate loads and corresponding displacements were within a few percent (about 1%) of each of the predicted values based upon the conventional rectangular stress distribution approach. Also the load-
Fig. 2.6 Typical Live Load vs. Midspan Deflection
displacement curves show that the single member had a final displacement of from 7 to 20 times the working load displacement at 80% to 118% of the calculated ultimate load on the section.

2.5 LOAD DISTRIBUTION OF THE MULTI-UNIT SYSTEM

One of the more important aspects of the structural performance of the system is the lateral load distribution characteristics. Since model-prototype correlations for the one half-scale models were previously established, a one half-scale model bridge deck was used to study the load distribution behavior. In order to consider the full range of loadings, the study was conducted in the following parts: load distribution characteristics for loads producing stresses in the elastic range; load distribution characteristics for loads producing stresses in the inelastic range.

Load Distributions Through the Elastic Range:

For the loadings producing concrete stressed in the elastic range, influence surfaces for deflection were developed. The bridge slab was sequentially loaded with an 8 kip load at five equally spaced locations along the center line of each box section. The displacement of each unit was measured at mid-span for each of the thirty load locations. Using Maxwell's reciprocal theorem, influence surfaces for deflection were obtained for a unit load applied to each box unit at mid-span. These surfaces are shown in Figure 2.7.

The Guyon-Massonnet load distribution theory was used to predict lateral moment and deflection distribution coefficients for comparison.
Fig. 2.7 Influence Surfaces for Deflection
This theory is based on converting the bridge deck to an equivalent orthotropic plate with equivalent average flexural and torsional stiffnesses. The problem is formulated by the following equation:

\[ D_x \frac{\partial^4 W}{\partial x^4} + 2H \frac{\partial^4 W}{\partial x^2 \partial y^2} + D_y \frac{\partial^4 W}{\partial y^2} = q \]

Where

- \( W \) - Transverse displacement
- \( D_x, D_y \) - Flexural stiffness
- \( H \) - Torsional stiffness
- \( q \) - Load intensity

The Guyon-Massonnet solution (Levy Form) to the formulation takes the following form:

\[ w = \sum_{m=1}^{\infty} W_m k_m \]

for a sinusoidal line load. \( W_m \) is the mean deflection found by equally distributing the line load over the entire width of the plate. Since the first terms of the solution for this particular loading produces 95% of total deflection, a good approximation can be found from this first term. As a result the following coefficient can be defined.

\[ K = \frac{W}{W_1} = \frac{W_1}{W} = k_1 \]

The distribution coefficient \( K \) can be used for comparison and the actual deflections need not be considered.

Values of \( K \) were analytically determined by considering a combination of flexural and torsional stiffnesses which best represented
the test results. The measured and predicted results are shown in Figure 2.8. The parameters \( \alpha \) and \( \theta \) are used in the prediction relations. The values of \( \theta \) are representative of the transverse stiffness considering an effective thickness of cast-in-place slab over the joint of the precast channels, while the values of \( \alpha \) are representative of the torsional stiffness of the composite box. As can be seen from the figure, the distribution characteristics change with the location of applied load. However, under the most critical condition, the distribution percentages are less than those for composite I-beam construction. It should also be noted that for load in the center region of the slab, the distribution characteristics are very similar to that of a monolithic plate with equal flexural stiffnesses in both orthogonal directions.

**Load Distributions Through the Inelastic Range:**

Since the bridge slab used in the study was to be loaded to failure for the purpose of evaluating the composite behavior of the member as the loads approached an ultimate value, the load arrangement was not ideal for load distribution studies. The loads used for this test series were one-third-point loadings to simulate the trailer wheels of the standard AASHO HS20-44 loading as previously described in Art. 2.2. There were eight loads in total with four centrally located in each lane of the two lane bridge deck. This condition simulated trailers located in the center of each lane and symmetrically about mid-span. The loading sequence was such that through the elastic range and approximately one-half way through the inelastic range each
Fig. 2.8 Deflection Distributions for Single Load
lane was loaded independently. In addition, a single trailer located at the center of the bridge slab was loaded in each sequence. Each of these three locations, plus both lanes simultaneously, was loaded in a series of increasing load levels ranging from 4 kips to 20 kips per ram.

Deflection values predicted by the Guyon-Massonnet theory for single concentrated loads were combined to produce predictions for the trailer wheel load combination. The values of \( \alpha \) and \( \theta \) previously established were used for the elastic range. The values predicted were compared to measured displacements and a good correlation was again observed. When considering the inelastic range, an equal flexural stiffness in each direction was used to determine \( \alpha \) and \( \theta \). This led to predicted results that best fit the experimental values. These results indicate that the most critical lateral load distribution condition exists in the elastic range as opposed to the inelastic range or at the ultimate load. These results are shown in Figure 2.9.

In addition to the comparison of the experimental displacements with those predicted by the Guyon-Massonnett theory, the measured distribution was compared with distribution coefficients recommended by the AASHO specifications. These results compare very well and are shown in Figure 2.9.

In general the lateral load distribution characteristics of the proposed system were very good. Furthermore, for all practical design purposes, the lateral distribution behavior of the proposed system can be considered equivalent to a monolithic multicelled box slab.
CENTER LINE OF BOX UNITS

MEASURED
- P = 4 KIPS, ONE LANE LOADED
- P = 20 KIPS, ONE LANE LOADED
- P = 4 KIPS, BOTH LANES LOADED

PREDICTED, \( \alpha = 1.0 \)
- COMBINATION OF \( \Theta = 0.9 \) & \( \Theta = 0.607 \)
- \( \Theta = 0.368 \) EQUAL STIFFNESS IN \( x \) & \( y \)
- UNIFORM DISTRIBUTION, i.e. 80% on each unit

FRACTION OF WHEEL LOAD (ONE LANE LOADED)
AASHO - CONCRETE BOX GIRDERs \( \text{We}/7.0 = 71.5\% \)
EXPERIMENTAL \( \text{We}/7.32 = 68.2\% \)

Fig. 2.9 Deflection Distribution for Model Wheel Load
2.6 FAILURE MODES OF SINGLE MEMBERS AND OF THE MULTI-UNIT SYSTEM

The failure modes of the single members were quite different from that of the multi-unit system. This difference was due to the loss of composite action of the single member which did not occur in the multi-unit system.

The loss of composite action in the single units was a result of the unconfined channel legs spreading, allowing excessive slips and slab separation at the time of failure. The actual failure occurred as a compression failure, or a combination of diagonal tension and compression failure in the upper portion of the channel legs. The same general mode of failure developed in all the single units tested in the program. The loss of composite action which resulted in the failures can be attributed to a wedging action of the shear connector designed to transfer the horizontal shear force between the two components of the member. Detailed descriptions of the connectors and the failure modes were presented in Interim Reports 68-8 and 68-9.

The multi-unit system failed in flexure with partial yielding of the prestressing strand and crushing of the composite slab. Prior to failure, tension cracking had progressed through the entire depth of the channel and into the lower region of the top slab over the channel legs. Since no discernible slips or slab separation had occurred through the failure load, it was apparent that complete composite action was maintained. It is of interest to note that the same type horizontal shear connectors were used for the single unit members and the multi-unit system. The difference in the effect of the connectors was due to the restraint of the channel legs by the adjacent channels. As a result, it
was concluded that the type of connectors used in the model bridge slab was adequate for the proposed bridge deck system. A detailed description of the failure mode of the model bridge deck was presented in Interim Report 68-14.
CHAPTER III
ECONOMIC EVALUATION OF THE PROPOSED SYSTEM

3.1 INTRODUCTION

Before any new or different structural system can be adopted it must exhibit either superior structural performance, increased economy, or both, over existing methods of construction. With equality of these two primary considerations for two structural systems changing from one system to the other would not be warranted. To determine the economy of the proposed system, an economic evaluation of the proposed composite box bridge deck system was undertaken.

3.2 METHOD OF EVALUATION

One of the methods of obtaining a realistic cost evaluation of the proposed bridge system would be to compare the cost of actual bridge decks designed using the proposed members with the conventional superstructures which have been recently constructed in Missouri. Actual prices were available for the comparison structures and estimated costs were only required for the proposed system.

Three typical bridge structures were used as bases of comparison. These structures were selected to give a representative range of span length and to consider the most common types of bridge superstructures used for shorter spans in the Missouri Highway system.

The first structure considered was a three span (34' - 34' - 34'), 26-foot roadway, precast slab structure designated by A-2141. The second was a continuous composite I-beam structure with 26-foot roadway and three
unequal spans (35' - 43' - 35') designated by A-2039. The third structure had four spans (43' - 70' - 70' - 43'), a 28-foot roadway, a voided cast-in-place slab deck section and was designated by A-2416. Each of these structures was designed with AASHO H15 loading. The cross sections of the superstructures of these three bridges are shown in Figure 3.1. The bridges considered here were approved for construction in late 1966 and 1967. The Missouri State Highway Commission made available actual unit costs on these three structures to be used in the cost analysis. Quantities were also obtained from the Highway Commission for these superstructures and combined with the unit costs to produce a final cost to be used in the comparison.

For the cost analysis, the superstructure of each of the three bridges was redesigned using the proposed composite box bridge deck system. The design procedure was similar to that used for the design of the test specimens with appropriate revisions. These revisions account for continuity with respect to live load over the supports at the interior bents, the recommended section revisions by the prestressed concrete producers, and the slab overhang recommendations made by the general contractors consulted in the study. The latter two of these revisions are discussed in the following articles.

The channel sections as they are cast act as simply supported members as discussed in Chapter II. The dead load of the channel and of the cast-in-place top slab are supported by the channels as simple beams. However, after the cast-in-place top slab has cured and if this top slab is continuous and reinforced over the supports, any additional
Fig. 3.1 Original Superstructures
loads applied to the deck will be resisted by the resulting continuous system. For the AASHO truck loadings considered in the redesigns, the design moments were determined for this continuous condition. The resulting cross sections of the redesigned bridge decks are presented in Figure 3.2.

The detailed design procedure for the live load moments at mid-span was the same as that for the dead load moments. On the other hand, the design procedure for the live load moments over the support required the consideration of a combined condition of a prestressed compression zone and an ordinary reinforced tension region. The prestressed compressive stresses were combined with compressive stresses resulting from the live load moments for consideration of the maximum allowable stress, while only the stresses due to the live load moment were used in determining the required tension reinforcing in this region. A detailed description of the design procedure and a revised computer program was presented in Interim Report 69-1.

3.3 CHANNEL COST ESTIMATION

The most economical prestressed concrete members are usually precast and pretensioned. The proposed composite box bridge deck system was designed utilizing this type of member. Since the channel portion of the proposed bridge deck is not drastically different from some commonly used precast pretensioned building members, prestressed concrete producers could be expected to estimate the production and delivery cost of the channel members with a reasonable degree of accuracy.
Fig. 3.2 Redesigned Superstructures
An initial design of the three bridge decks to be considered was made sufficiently detailed to allow a realistic cost estimate of production. Several precast prestressed concrete producers in Missouri, Kansas, Iowa and Nebraska were contacted and asked to make cost estimates for the members resulting from the initial design. At the same time these producers were asked to make recommendations for any changes to the channel section or method of production.

Three of the "larger volume" prestressed concrete producers responded to the request with both estimates and recommendations. The member prices obtained from the producers are listed in Table 3.1 on a square foot basis. These price estimates were made independently by each producer from the same information with no knowledge of the cost estimate of the other producers. In addition to the particular channel cross section, prestressing strand, and the span length, the producers were asked to consider a reasonable volume of production, rather than estimating on the basis of custom members. Since the variation in the price estimates for each of the members considered was small and since the estimates were independent, the values obtained should be quite reliable.

The cost estimates furnished by the producers were made for the members delivered within a one hundred mile radius of the plant. In addition to including this transportation cost as part of the initial estimate, some additional transportation costs were furnished by one of the producers and are presented in Table 3.2.

From the initial conception of the proposed system, it was anticipated that the most economical method of production was the casting of the channels in an inverted position. However, the first
### Table 3.1

**COST PER SQUARE FOOT OF PRECAST-PRESTRESSED CHANNELS**

<table>
<thead>
<tr>
<th>Producer</th>
<th>Casting Method</th>
<th>Depth/Length</th>
<th>20&quot;/33'</th>
<th>26&quot;/34'</th>
<th>26&quot;/42'</th>
<th>34&quot;/42'</th>
<th>34&quot;/69'</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 1</td>
<td>Inverted</td>
<td></td>
<td>$3.15</td>
<td>$3.50</td>
<td>$3.75</td>
<td>$4.26</td>
<td>$4.51</td>
</tr>
<tr>
<td>No. 1</td>
<td>Upright</td>
<td></td>
<td>3.25</td>
<td>3.60</td>
<td>3.50</td>
<td>4.05</td>
<td>4.10</td>
</tr>
<tr>
<td>No. 2</td>
<td>Upright</td>
<td></td>
<td>3.15</td>
<td>3.54</td>
<td>3.46</td>
<td>3.71</td>
<td>4.03</td>
</tr>
<tr>
<td>No. 3</td>
<td>Upright</td>
<td></td>
<td>3.21</td>
<td>3.52</td>
<td>3.81</td>
<td>4.45</td>
<td>4.51</td>
</tr>
</tbody>
</table>

### Table 3.2

**TRANSPORTATION COSTS**

<table>
<thead>
<tr>
<th>Mileage from Plant to Job Site</th>
<th>Cost Per Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>$34.00</td>
</tr>
<tr>
<td>50</td>
<td>50.00</td>
</tr>
<tr>
<td>100</td>
<td>77.00</td>
</tr>
<tr>
<td>150</td>
<td>104.00</td>
</tr>
<tr>
<td>200</td>
<td>130.00</td>
</tr>
<tr>
<td>250</td>
<td>157.00</td>
</tr>
<tr>
<td>300</td>
<td>184.00</td>
</tr>
</tbody>
</table>
recommendation of two of the three producers responding to the request was that it would be more economical to cast the channels in the upright position as a result of high handling and inverting costs. The third producer was contacted and requested to provide estimates for both methods of production. As can be seen from the table, these prices vary such that the inverted casting method is more economical in the shorter spans and the upright casting method more economical in the longer spans considered.

It was concluded from this information that the upright method of casting should be used for channel production. This method of production allows a desirable change in the horizontal shear connector arrangement. Rather than requiring a welded connector as originally proposed, a more conventional connector of U-shaped reinforcing bars extending from the top of the precast channel legs is recommended. The latter type connectors are conventionally used to develop composite behavior in standard precast prestressed I-beam bridge construction and were not considered in this study.

The second recommendation from the producers was an alteration to the channel section and was of much less consequence than the method of production. The changes were the radius at the base of the channel legs and the slope on the inside of the legs. These changes were made to reduce stress concentrations at the intersection of the legs and the base of the channels, to reduce the channel dead load, and to aid in casting. These changes were incorporated into the design presented in Interim Report 69-1 and can be seen by comparing the channel sections of the test specimens (Figure 2.1) and those of the redesigned bridge decks (Figure 3.2).
3.4 CONSTRUCTION COST ESTIMATION

In order to complete a cost analysis of the proposed composite box bridge deck system, an evaluation of the on-site construction cost was required. To accomplish this in a realistic manner, two general contracting companies with considerable experience in bridge construction were consulted. Since detailed plans were not available for an estimate to be made directly by the contractors, they were asked to furnish unit costs and methods of estimation for the proposed system. As would be expected, each of the companies consulted used methods of estimating that were slightly different and resulted in cost estimates with some variation. On the other hand, many of the unit prices and labor estimates were very consistent even though each company's effort was independent and without the knowledge of the other's work.

Each of the companies considered the following items in the cost estimations:

1. Erection cost which included both labor and equipment.
2. Forming costs for both the slab overhangs and the curb and parapet. This item contained forming material as well as labor rates for the forming. The forms also include a working space outside the slab overhangs.
3. Concrete, including the labor of casting as well as the material cost.
4. Overhead, equipment, insurance and supervision.

In addition, reinforcing steel was considered in the estimates; however, the Bridge Division of the Missouri Highway Commission furnished unit prices for this item which included material and placement labor.
The contractors were also consulted regarding the completeness of the estimate. Each indicated that this procedure would provide as complete an estimate as would be possible without detailed design drawings and knowledge of the locations of the structures.

A complete listing of the two estimation methods was presented in Interim Report 69-1. In general the material and labor costs were the same for both methods. The three principal differences between the two estimation methods were in the erection of the precast prestressed channels, forming of the slab overhang, and overhead. The differences resulting from the two methods of estimating partly account for the variation in superstructure prices presented. However, this variation is not large enough to discredit either estimation method but should be expected since both contractors did not make the estimate but simply furnished unit prices and methods.

3.5 SUMMARY OF SUPERSTRUCTURE COSTS

To complete the economic evaluation of the proposed composite box bridge system, the channel costs and construction costs were combined for the three redesigned bridge superstructures. These combined costs were compared with the cost of the original superstructures in order to make the evaluation.

These combined costs were subsequently broken down into total cost and percentages of the total cost according to the following divisions:

1. Overall costs including comparison with cost of the original structure.
2. Percent for channel sections.
3. Percent for on-site construction materials.
4. Percent for construction labor.
5. Percent for equipment, overhead and insurance.

Since four estimates for each channel section and two estimation methods were available, several total cost figures are possible. For comparison purposes high, low, and average values were considered. These total costs and percentages are presented in Table 3.3.

A comparison between the cost of the bridge superstructures, as originally designed and constructed by the state and as redesigned using the proposed structural system, can be made by considering the cost per square foot prices listed in Table 3.3. Using the average estimated values, it can be seen that for the shorter span bridges, A-2141 and A-2039, the cost of the original superstructures exceeds the estimated cost of the proposed system by 10.2% and 12.7%, respectively. These values would indicate that, even with the conservative nature of the estimate, the proposed system does not appear to be significantly more economical than present types of construction. However, the estimated cost of the proposed system for bridge A-2416 is about 21.4% less than the actual cost of the voided slab original superstructure. It appears that for medium span ranges the proposed system has an economic advantage.

Several additional observations can be made from the values presented in Table 3.3. An examination of the variation between high and low values shows that the difference increases with span length. For the two shorter span structures the variation was approximately 6.5% of the average total cost, while this value increased to 16.0% for the longer span bridge. The cost of the channels makes up a large portion
<table>
<thead>
<tr>
<th>Bridge</th>
<th>Cost of Superstructure</th>
<th>Saving</th>
<th>Chann. Cost (%) of total</th>
<th>Mat'l's (%) of total</th>
<th>Labor (%) of total</th>
<th>Equipment &amp; Overhead</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Proposed System Total</td>
<td>$/sq.ft.</td>
<td>State Total $/sq.ft.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A-2141</td>
<td>High</td>
<td>$19,882.40</td>
<td>6.83</td>
<td>7.38</td>
<td>+0.55</td>
<td>41.8</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>19,284.23</td>
<td>6.63</td>
<td>7.38</td>
<td>+0.75</td>
<td>42.1</td>
</tr>
<tr>
<td></td>
<td>Low</td>
<td>18,760.58</td>
<td>6.45</td>
<td>7.38</td>
<td>+0.93</td>
<td>42.5</td>
</tr>
<tr>
<td>A-2039</td>
<td>High</td>
<td>24,820.91</td>
<td>7.44</td>
<td>8.22</td>
<td>+0.78</td>
<td>41.5</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>23,956.68</td>
<td>7.18</td>
<td>8.22</td>
<td>+1.04</td>
<td>41.8</td>
</tr>
<tr>
<td></td>
<td>Low</td>
<td>23,156.60</td>
<td>6.94</td>
<td>8.22</td>
<td>+1.28</td>
<td>42.2</td>
</tr>
<tr>
<td>A-2416</td>
<td>High</td>
<td>58,756.60</td>
<td>8.36</td>
<td>10.33</td>
<td>+1.97*</td>
<td>51.2</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>54,376.58</td>
<td>7.74</td>
<td>10.33</td>
<td>+2.59*</td>
<td>52.1</td>
</tr>
<tr>
<td></td>
<td>Low</td>
<td>50,088.78</td>
<td>7.13</td>
<td>10.33</td>
<td>+3.20*</td>
<td>52.8</td>
</tr>
</tbody>
</table>

* For bridge A-2416, the cost of the superstructure was increased by $0.38/sq.ft. Actual savings would be

  
  High \( $1.97 - 0.38 = +$1.59/sq.ft. \)
  
  Average \( $2.59 - 0.38 = +$2.21/sq.ft. \)
  
  Low \( $3.20 - 0.38 = +$2.82/sq.ft. \)

**TABLE 3.3 BASIC COST BREAKDOWN**
of the total cost and accounts for a large part of the variation. The variation in channel costs was seen in Table 3.1 to increase with span length because of handling and casting problems. Due to the increased roadway width six channels were used for the longer span structure rather than five members as was used for the shorter span bridges. The combined effect of an increased number of members with increased channel cost variation for the longer span bridge largely accounts for the increase in total cost variation.

In addition, it can be seen from Table 3.3 that the channel cost accounts for about 45% of the total superstructure cost. The estimates for these members were made without the benefit of production experience and would be expected to reduce as this experience is gained. Because of the nature of the system, a reduction in the square foot cost for the channels results in a decrease in the square foot cost for the superstructure of about 87% of the reduction. Consequently, a reduction in channel cost would correspondingly effect the total cost of the structure.

Also the distribution of costs, as expressed by the percentages of the total superstructure cost, should be considered. For most bridge construction a minimum of 30% to 40% of the total cost of the structure is on-site labor cost, as compared to 10% to 15% for the proposed system. As a result, variations in the labor market and variations between local labor wage scales should have a light effect upon the cost of the structure.
In addition to the H15 loading, all three superstructures were designed for an HS20 loading. This resulted in an increase of cost averaging 1.73%. The cost results from the HS20 loading are given in Table 3.4 along with the H15 results for comparison.

Table 3.4

COST PER SQUARE FOOT FOR H15 LOADING AND HS20 LOADING

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Bridge</th>
<th>Bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A-2141</td>
<td>A-2039</td>
</tr>
<tr>
<td>H15-44</td>
<td>HS20-44</td>
<td>H15-44</td>
</tr>
<tr>
<td>High</td>
<td>6.83</td>
<td>7.00</td>
</tr>
<tr>
<td>Average</td>
<td>6.63</td>
<td>6.76</td>
</tr>
<tr>
<td>Low</td>
<td>6.45</td>
<td>6.56</td>
</tr>
</tbody>
</table>

An item which cannot be considered from the information presented in this chapter and in Interim Report 69-1 is the reduced construction time which is possible with the proposed bridge system. An evaluation of the effect of construction time involves many factors which are known only after the bridge site has been selected and the construction is considered along with the total highway project. However, there are many times when the construction of a bridge, or bridges, is critical to the completion of a project, whereby the construction time becomes an economic consideration.
CHAPTER IV

DESIGN CONSIDERATION

4.1 INTRODUCTION

The bridge system proposed in this study can be designed without any special considerations different from those normally considered in a bridge designed of precast prestressed concrete. However, there are features of the proposed system which warrant discussion. The items for which discussions are presented can be categorized as follows: 1) Recommended cross-section dimensions and variations, 2) Recommended design considerations. The more important features of the design of the proposed system are presented in order that the designer can use the system with confidence.

4.2 RECOMMENDED STANDARD CROSS-SECTION DIMENSIONS AND CHANNEL PLACEMENT

In order to receive the greatest economic benefit from the proposed system, limitations on cross-sectional variations and member spacings are given.

1.) Cross-Sectional Variation: It is recommended that a cross-section based on the dimensions given in Figure 4.1 be used for all channel sections with span lengths from 30 to 80 feet. The only change in the dimensions of the cross-section used to resist various bending moments is that of depth. As noted in Figure 4.1 the recommended variation in depth is limited to 4 inch steps with possible channel heights of 20", 24", 28", 32" and 36". In addition, with fixed dimensions of the base and a constant slope on the inside of the channel legs,
MONOLITHICALLY CAST-IN-PLACE TOP SLAB

CORRUGATED STEEL VOID FORM

PRECAST CHANNEL

Fig. 4.1 Recommended Standard Cross-Sections
a single set of forms can be used to cast several depth members, which increases the economy of the system.

2.) Void Forms: It is recommended that a single standard corrugated metal arch void form be used. The arch forms should be attached to the channel legs at the supports and with an adequate corrugation thickness combination to support workmen during construction. (An adequate combination found from the experimental work was corrugations of 3 1/2" pitch, 7/8" height and 26 gage thickness).

3.) Variation in Bridge Deck Width: Since the structural performance of the proposed system is dependent upon the channel legs being restrained in the transverse direction, it is recommended that the channels be placed side by side with the variation in width of the bridge deck to be taken up by overhangs at the exterior channels. This arrangement is shown in Figure 3.2. In addition, it is recommended that the overhang be limited to a minimum of six inches and a maximum of three feet. When the overhang exceeds three feet, an additional channel can be added to reduce the overhang to within the recommended limits.

4.3 RECOMMENDED DESIGN CONSIDERATIONS

The purpose of this study was not to develop new and different design procedures, but rather to check the proposed system with respect to existing procedures and practices. As a result, the proposed system can be designed according to existing AASHO specifications. However, for the convenience of the designer the following items and additional recommendations are presented.

1.) Channel Design: Standard prestressed-pretensioned design procedures can be used for design of the channels. The channel design should be based on a simple support for the dead load of the channels
and the cast-in-place top slab, with a continuous support for all live load received by the structure.

2) Design of Connection at Interior Bents: Consideration of the problem of positive moments at the interior supports due to creep and shrinkage of the precast prestressed member was not within the scope of the experimental phases of the study presented here. However, the problem does exist and should be considered in the design of the proposed system. Methods for the design of continuity reinforcing for both positive and negative moments at the interior supports have been studied by other researchers. Methods were developed for continuous I-beam bridges (5) and then extended to other bridge members (6). It is recommended that this procedure be applied to the design of the continuity connection at the interior supports of the proposed system.

3) Horizontal Shear Connector Design: The shear connector design and configuration presented in the study were based on casting the channels in the inverted position. The performance of these connectors was adequate; however, it was shown to be most economical to cast the channels in the upright position, and as a result standard composite design procedures and connectors can be used for horizontal shear.

4) Design of Top Slab Transverse Reinforcing: The design of the top slab reinforcing presented in the interim reports considered a transverse strip of the slab as a haunched beam. For the single units the haunched beam was considered to be simply supported and for the multi-unit bridge deck the beam was considered to be continuous over
the supporting channel legs. This procedure would appear to be very precise; however, after comparison of design moments it was concluded that the results found by the standard AASHO design procedure differ from those found by the haunched beam method only slightly. As a result, it is recommended that the standard AASHO procedure be used to obtain moments for design of the top slab of the proposed system. It should also be pointed out that the actual depth of the section being considered should be used for design. In addition, due to the variation in depth, the transverse reinforcing can be straight and still provide the necessary reinforcing in both the positive and negative moment regions.

5) **Reinforcing at Diaphragms:** Transverse reinforcing at each diaphragm and at supports should be used according to present convention. Due to the method of forming diaphragms (terminating the void for the width of the diaphragm), it is recommended that the reinforcing in the diaphragms either pass through the interior channel legs or be anchored to the legs in a manner similar to that used for precast-prestressed I-beam bridge construction.

5) **Long Time Deflections:** Since the long time deflections of any concrete beam, non-prestressed or prestressed, are unpredictable, it is recommended that the design of the channel members be such that the deflections under sustained loads are zero or upward. Theoretically a concrete member with zero deflection (the zero displacement is based upon the initial horizontal member prior to the transfer of the prestressing force) will have a zero curvature and, as a result, only axial shortening due to creep.
As initially stated, the proposed bridge system can be designed in accordance with the existing AASHO specifications with satisfactory results. The considerations and recommendations presented in this chapter are intended to be an assistance to the designer rather than additional conditions necessary for adequate structural performance of the bridge.
CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

Based on the results of the study as presented in the interim project reports and summarized in this final project report, certain conclusions can be drawn. The conclusions made here are very general in nature. For more detailed information and conclusions the reader is referred to the interim project reports (1, 2, 3, 4).

1. The structural performance of the proposed system was predictable by conventional prestressed and reinforced concrete theories for both the working range and at ultimate.

2. The system was more economical than the comparison structures, irrespective of the type of construction and span length of those considered. The percent decreases in superstructure cost based on an average estimated value ranged from 21.4% for the 43'-70'-70'-43' voided slab bridge to 10.2% for the 34'-34'-34' precast slab bridge. In addition the cost is essentially unaffected by a change from H15-44 to HS20-44. The average change in superstructure cost between the two loading conditions was approximately 1.7%. This comparison did not consider other types of precast prestressed construction, primarily since they are not in use in Missouri. However, based on the cost differential between the proposed system and those considered for comparison, this system should at a
minimum, be competitive with conventional precast prestressed I-beam and box-girder bridge construction.

3. There are limitations of economical span lengths for the proposed bridge system. These limitations are determined primarily from deflections of the channels when subjected to the dead load of the cast-in-place top slab, and from limitations fixed from transportation restrictions. The most feasible span range appears to be from 30 feet to 80 feet for the recommended section depths.

4. Even though the experimental studies were conducted with all normal weight concrete, there were no results which would indicate that light weight concrete could not be used for either the channel or the cast-in-place top slab. The time-displacement results (1) indicated a minimum change in displacement due to curing of the top slab, which in turn indicates a minimum effect of shrinkage. Provided low shrinkage, low creep concrete is used, either or both components of the proposed system could be constructed from light weight concrete.

5.2 RECOMMENDATIONS

The following recommendations are made based on the results of the study being summarized in this report.

1. It is recommended that two trial structures be constructed using the proposed system. The first should be in the 30'-40' span range with the second being
in the 70' - 80' span range. Studies of the actual
cost for these structures should be made
to substantiate the economy indicated by the cost
estimates of the economic evaluation phase of this
project. In addition, a limited short and long time
deflection study of the trial structures should be
conducted.

2. Since the deflections due to sustained loads are critical
to the design of the proposed system, it is recommended
that a limited study be conducted to examine methods of
controlling deflections through temporary supports. It
is probable that the member size and cost can be reduced
for systems which are designed with a limited number of
temporary supports during placement and curing of the
cast-in-place top slab.

3. Due to the span length limitation resulting from trans­
portation restrictions and the increased span length
requirements for overpass structures, it is recommended
that a study be conducted to determine the feasibility
of splicing the channel members in the positive moment
region. This splicing capability would allow precast
channels to be used without the transportation or handling
limitations which exist at this time.
REFERENCES


