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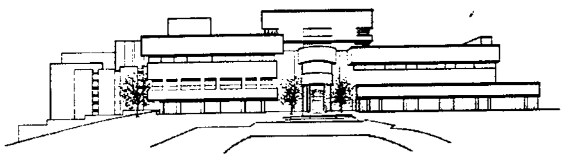

College of Engineering
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Development of a New Concept For Florida's Bridges

Vol I

R. Sen, S. Stroh, J. Olbinska, S. Hassiotis and G. Mullins
Department of Civil and Environmental Engineering

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CONVERSION FACTORS, US CUSTOMARY TO METRIC UNITS

Multiply	*by*	*to obtain*
inch	25.4	mm
foot	0.3048	meter
square inches	645	square mm
cubic yard	0.765	cubic meter
pound (lb)	4.448	newtons
kip (1000 lb)	4.448	kilo newton (kN)
newton	0.2248	pound
kip/ft	14.59	kN/meter
pound/in^2	0.0069	MPa
kip/in^2	6.895	MPa
MPa	0.145	ksi
ft-kip	1.356	kN-m
in-kip	0.113	kN-m
kN-m	0.7375	ft-kip

PREFACE

The investigation reported was funded by a contract awarded to the University of South Florida, Tampa by the Florida Department of Transportation with Parsons, Brinckerhoff, Quade & Douglas, Inc. Tampa as consultants. Mr. Larry Sessions served as Project Manager for FDOT. We thank Mr. Sessions for his interest in the study.

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The final report was prepared with the assistance of graduate students Mr. G. Atluri, Mr. J. Puvvala and Mr. R. Donaldson. Their contribution is gratefully acknowledged.

EXECUTIVE SUMMARY

This report presents the findings from the first year of a three-year investigation to develop a new bridge superstructure concept for spans varying between 200-400 ft.

A state-of-the art survey was carried out to compile information on new materials, technologies and also to obtain examples of recently constructed innovative bridges. To identify deficiencies in existing Florida bridges, the maintenance records of several representative bridges *(all in Districts 1 and 7)* were reviewed. Cracking of the deck slab was found to be the most common problem.

A careful evaluation was made of over twenty concepts proposed in the literature by several of the Tampa Bay region's most prominent long span bridge designers. Four concepts were short-listed and a double composite concept selected for detailed study. In this concept, an additional bottom slab is cast in the negative moment region in continuous plate girder or box bridges (*the two most widely used sections in Florida*) to ensure composite action in the negative moment region as well. This leads to increased strength, stiffness and stability. This concept has not been used in the United States though it has proven to be successful in Europe.

The double composite concept was evaluated using AASHTO LRFD specifications. An optimized conventional plate girder design i.e. composite slab for positive moment regions, was used as a benchmark. Cost savings for the comparable double composite section were 6% and structural steel weight savings were 12%. Similar savings were also realized for box girder sections. Greater savings of 8% and 14% respectively were realized for hybrid girders utilizing high performance steel flanges and regular steel webs.

The first phase of the study has demonstrated the potential of the double composite concept. The aim of the second phase is to conduct additional design studies, laboratory tests and numerical analyses to further refine and develop the concept. Proposed experimentation includes specific tests for developing new design criteria followed by a scale model test of a three-span hybrid double composite plate girder model implementing the criteria. To maximize economy, girder spacing will be kept as large as practical and alternate mechanisms for shear transfer (*other than stud connectors*), investigated. The elastic and ultimate response of the model will be assessed.

An additional phase is recommended for the future to allow monitoring of stresses in a demonstration prototype structure. Such monitoring could be carried out in cooperation with the Structures Research Office.

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1. INTRODUCTION

1.1 Introduction

In April 1997, the Florida Department of Transportation issued a RFP (request for research proposal) entitled *"Development of an Alternative Long Span Bridge Design System*". The stipulated goal of the project was to develop a new segmentally constructed composite bridge superstructure system that would be appropriate for long span bridges built in Florida.

The three-year project was divided into two phases. In the first phase, a new superstructure concept was to be developed. In the two-year second phase, this concept was to be further developed on the basis of laboratory testing and numerical analyses. The total funding for the project was $280,000 with Phase I funding limited to $80,000.

The University of South Florida with Parsons, Brinckerhoff Quade & Douglas, Inc., Tampa as consultants were selected to conduct the research. Work commenced on the study on January 1, 1998. This report summarizes the findings from Phase I that were concluded in December 1998.

1.2 Objectives

The overall objective of the study was to develop a composite bridge superstructure suitable for long span bridges defined as spanning between 200 to 600 ft. According to the RFP, the composite design had to utilize the "best properties of structural steel, prestressing steel, concrete, carbon fibers and other materials". In addition, the new bridge design system had "to possess characteristics of both cost efficient and good aesthetic design" and "utilize materials in a optimal manner with least weight". Moreover, the researchers were required to consider "a post-tensioned, space frame truss combined with a composite overtopped steel grid or aluminum deck as one possible option". Finally, the researchers were required to "identify deficiencies in existing knowledge and indicate areas requiring improvement".

1.3 Approach

To meet the objectives of this study, an in-depth literature search was carried out to obtain information on the disparate topics that required to be examined. In addition, personal contacts were made with selected Department of Transportation offices in the United States and highway agencies overseas to obtain additional information.

To provide an appropriate contextual framework for the literature search, computerized searches were made of the National Bridge Inventory (NBI) database. As this contained data until December 31, 1996 a further search was carried out on Florida Department of Transportation's Bridge Inventory Database (BID) that had records for an additional year. Together, these databases provided information on all the bridges in the 200-600 ft span range that were existence in the nation.

Analysis of the bridge inventory data (presented in the next chapter) showed that over 97% of the state's bridges had spans between 200-400 ft with less than 3% falling between 400-550 ft. There were no bridges with spans between 550-600 ft. Moreover, structures in the 400-550 ft range were landmark structures. The data also revealed that steel bridges were not losing their market share; over the 5-year period between 1991-95, steel's share of long span bridges nationwide was 69.8% compared to 82.3% in Florida. This suggested that competition between concrete and steel alternates was very much in evidence.

Partly as a result of these findings and also the need to bring the project to a sharper focus given its limited time frame, several of the original objectives were revised or dropped altogether during the course of the study. Thus, the upper limit for span was reduced from 600 ft to 400 ft and there was no insistence on a detailed consideration of the post-tensioned space truss bridge due to constructability issues (see Chapter 12).

The results of the literature survey provided several innovative concepts linked to the use of new materials. Although Parsons Brinckerhoff were consultants for the study, a review panel was set-up to sift through these innovative concepts. Members of the review panel included some of the most experienced long span bridge designers in the Tampa Bay area. The intent was to ensure that the process by which decisions were arrived at least took advantage of all the expertise that was locally available.

A total of twenty one concepts proposed in the literature were reviewed. In addition, four others proposed at the meeting of the review panel were given serious consideration. The most promising concepts were short-listed for further consideration and an eventual selection made of the double composite concept that was the subject of more detailed analysis and design. In the detailed consideration, comparative designs were carried to determine the extent to which savings would be realized if the double composite concept were used. The designs were all based on AASHTO LRFD specifications and SI units were used for the calculations.

The selection of the double composite concept for Phase II was undoubtably influenced by time and budget limitations in this project. With two years and a very limited overall budget, radical solutions that required complex fabrication or long term durability type evaluation were not considered suitable. Instead, ideas that have been successful elsewhere were given more serious consideration. Such evolutionary change was expected to find readier acceptance in the bridge community.

1.4 Organization of Report

The report contains 14 chapters and 7 appendices. For convenience, references cited are listed at the end of the chapter and all the appendices are included in a separate volume.

An analysis of bridge inventory data on long span bridges is summarized in Chapter 2. Recent advances have led to the development of High Performance Concrete (HPC) and High Performance Steel (HPS) that are likely to be increasingly used in the future. Brief descriptions of these materials are presented in Chapters 3 and 4 respectively.

Dead load is a very important consideration for longer span bridges. Consequently, information on measures that can lead to a reduction in dead weight are included in Chapters 5-7. Chapter 5 contains information on a new aluminum deck system that is available. Chapter 6 describes lightweight concrete that has been used successfully in Florida for over 35 years. Chapter 7 contains descriptions of various proprietary deck systems that are available. These include grid, exodermic, Harspan PT and lightweight decks made using advanced composites.

An area where innovation is likely in the future is in the use of "smart" technology. An essay describing some recent applications is contained in Chapter 8. In order to design and construct better bridges, information on common bridge deficiencies is very important. Chapter 9 is a report on problems commonly encountered in some long span bridges that are in the jurisdiction of Districts 1 and 7. This information was painstakingly gathered by reviewing all the available inspection records that spanned 17 years. Supplementary information on the deficiencies detected is contained in Appendices A and B.

Information on innovative bridges that have been built in the United States and elsewhere is presented pictorially in Chapter 10 with brief descriptions of the concept. Information on a total of 21 bridges is presented. This contains examples of the application of corrugated webs, extradosed bridges, stress ribbon bridges and other examples showing the use of new materials or innovative construction.

A brief summary of the 25 innovative superstructure concepts that were examined by the review panel is contained in Chapters 11 and 12. Chapter 11 contains information on the innovative concepts obtained from the literature survey. Chapter 12 describes the review process that led to the adoption of the double composite concept proposed in this study.

Preliminary analyses and design carried out to evaluate the double composite concept are contained in Chapter 13. Design criteria used in the evaluation is summarized in Appendix C. Four appendices, D-G provide calculation details for the four designs that were carried out. These were (1) conventional (single) composite design (Appendix D), (2) Double composite plate girder design (Appendix E), (3) Double composite box girder design (Appendix F) and (4) Double composite hybrid girder design (Appendix G). Recommendations on studies that may be suitable in Phase Two are discussed in Chapter 14.

2. SURVEY OF NATION'S BRIDGES

2.1 Introduction

The National Bridge Inventory (NBI) is a database containing information on bridges with spans exceeding 20 ft. This database was accessed for data on the nation's bridges spanning 200-600 ft. The data obtained in Jan. 1998 was current as of Dec. 31st 1996. In view of this, the Florida Department of Transportation were separately contacted. FDOT's Bridge Inventory Database (BID) contained information that was current as of Dec. 31st 1997.

This chapter analyzes information obtained from both the national and Florida databases to help identify underlying changes in bridge design and construction over the past 40 years. Sections 2.2-2.3 provide background information on the NBI database. Analysis of national data is summarized in Sections 2.4-2.8. and those for Florida's bridges in Sections 2.9. The principal conclusions from the analysis are contained in Section 2.10.

2.2 NBI Data

Data received from NBI was in the form of an ASCI file containing 116 items of information on each bridge. A complete description of these items may be found in FHWA's Recording and Coding Guide [2.1]. For the purposes of this study, only six items were required. The item numbers and their identification in Ref. 2.1 are summarized in Table 2.1.

Table 2.1 NBI Data Items Used [2.1].

Item No	Description
1	State Code
27	Year Built
43A	Kind of Material/Design
43B	Type of Design/Construction
48	Maximum Span Length
49	Structure Length

2.3 Bridge Types

The ASCI file obtained from FHWA contained information on a total of 3,776 bridges made of reinforced concrete, prestressed concrete and steel. Only 3,741 of these records contained information relating to the six items listed in Table 2.1. The remaining 35 had missing items and were disregarded in the analyses reported in Sections 2.4-2.9.

The NBI database has provision for 23 different bridge types. The codes identifying these bridge types and their description are summarized in Table 2.2.

Table 2.2 Bridge Types Identified by NBI.

Code	Description
01	Slab
02	Stringer/Multi-Beam or Girder
03	Girder and Floorbeam System
04	Tee Beam
05	Box Beam or Girders – Multiple
06	Box Beam or Girders - Single or Spread
07	Frame
08	Orthotropic
09	Truss – Deck
10	Truss – Thru
11	Arch – Deck
12	Arch – Thru
13	Suspension
14	Stayed Girder
15	Movable Lift
16	Movable Bascule
17	Movable – Swing
18	Tunnel
19	Culvert
20	Mixed Types
21	Segmental Box Girder
22	Channel Beam
00	Other

2.4 National Overview

Until quite recently, steel was the only material that could be used in long span bridge construction. However, developments in reinforced and prestressed concrete technology, particularly during the latter part of this century, have led to an increasing numbers of concrete bridges.

Table 2.3 shows the distribution of reinforced concrete, steel and prestressed concrete bridges in the country as of Dec 31st 1996. Of the 3,741 bridges, almost 83% are made of steel, 10% of prestressed concrete and 7% of reinforced concrete. The percentage of prestressed bridges is a measure of the loss of market share of steel over the last 40 years.

Table 2.4 summarizes information on the average age of the bridges. Not surprisingly, prestressed concrete bridges are the youngest. Reinforced concrete structures have the highest age indicating that few of the newer bridges are built using this material.

Table 2.3 Breakdown by Material Type.

Type	Number	Percentage
Concrete	259	6.9
Steel	3,117	83.3
Prestressed Concrete	365	9.8
Total	3741	100

Table 2.4 Average Age by Material Type.

Type	Period	Number	Average Age (yr)
Concrete	1907-95	259	45
Steel	1868-96	3,117	38
Prestressed	1959-96	365	9

2.5 Distribution by Span

As the focus of the research is on bridges spanning 200-600 ft it is instructive to examine the distribution of bridges that fall in this range. For this purpose, spans were considered in 50 ft increments and the numbers of reinforced concrete, steel and prestressed concrete bridges in each range determined. Additionally, percentages for each span range and their cumulative values were also computed for the three materials.

Table 2.5 provides a summary of the results. Inspection of Table 2.5 shows that 60% of all bridges between 200-600 ft have spans less than or equal to 250 ft. *Over three quarters of the bridges have spans less than 300 ft and only 10% of the bridges have spans above 400 ft.* For reinforced concrete, almost 92% of the bridges have spans below 400 ft. The corresponding percentages for steel is about 89% and for prestressed concrete 96%.

The relatively small percentage of bridges between 400-600 ft suggest that the focus of a generic solution, such as considered in this study, may be more appropriate for spans in the 200-400 ft. range.

Table 2.5 Distribution by Span and Material Type.

Span ft	RC		Steel		PC		Total	
	#	Cumulative	#	Cumulative	#	Cumulative	#	Cumulative
200-250	154 (59.5%)	154 (59.5%)	1819 (58.4%)	1819 (58.4%)	281 (77.0%)	281 (77.0%)	2254 (60.3%)	2254 (60.3%)
251-300	44 (17.0%)	198 (76.4%)	505 (16.2%)	2324 (74.5%)	50 (13.7%)	331 (90.7%)	599 (16.0%)	2853 (76.3%)
301-350	23 (8.9%)	221 (85.3%)	259 (8.3%)	2583 (82.3%)	10 (2.7%)	341 (93.4%)	292 (7.8%)	3145 (84.1%)
351-400	16 (6.2%)	237 (91.5%)	187 (6.0%)	2770 (88.9%)	10 (2.7%)	351 (96.2%)	213 (5.7%)	3358 (89.8%)
401-450	8 (3.1%)	245 (94.6%)	142 (4.6%)	2912 (93.4%)	8 (2.2%)	359 (98.4%)	158 (4.2%)	3516 (94.0%)
451-500	4 (1.5%)	249 (96.1%)	73 (2.3%)	2985 (95.8%)	2 (0.5%)	361 (98.9%)	79 (2.1%)	3595 (96.1%)
501-550	6 (2.3%)	255 (98.5%)	85 (2.7%)	3070 (98.5%)	1 (0.3%)	362 (99.2%)	92 (2.5%)	3687 (98.6%)
551-600	4 (1.5%)	259 (100%)	47 (1.5%)	3117 (100%)	3 (0.8%)	365 (100%)	54 (1.4%)	3741 (100%)

2.6 Distribution by State

The 3,741 bridges in the NBI database were analyzed to provide information on long span bridges in the 50 states, the District of Columbia and Puerto Rica. Table 2.6 summarizes the results. As data on Florida bridges is more current than those for the rest of the nation, comments relating to Florida's bridges are included elsewhere (see Sections 2.10-2.14). The following observations may be made based on the information contained in Table 2.6:

Reinforced Concrete - The largest number of reinforced concrete bridges are in Maine (51), followed by California (37) and Kansas (26). In percentage terms, these numbers reflect 50.5% (Maine), 11.8% (California) and 28.6% (Kansas). Sixteen states do not have any reinforced concrete bridges. These are Alaska, Connecticut, Indiana, Michigan, Montana, Nebraska, Nevada, New Hampshire, New Mexico, North Dakota, Rhode Island, South Dakota, Utah, Vermont, Wisconsin and Wyoming.

Steel - The largest number of steel bridges are in Pennsylvania (224), followed by New York (221), Washington (181) and Illinois (173). However, there are eight states where all the long span bridges are made of steel. These are Montana, New Hampshire, New Mexico, Rhode Island, South Dakota, Utah, Vermont and Wyoming.

Prestressed Concrete - The largest number of prestressed bridges were in California (176), followed by Georgia (28) and Colorado (15). In percentage terms, these numbers reflect 56.2% (California), 44.4% (Georgia) and 30.6% (Colorado). Nineteen states and DC do not have any prestressed concrete bridges. These are Arkansas, Delaware, Iowa, Kansas, Maine, Maryland, Massachusetts, Missouri, Montana, New Hampshire, New Jersey, New Mexico, Oklahoma, Rhode Island, South Dakota, Utah, Vermont, West Virginia and Wyoming.

Number of Bridges - California with 313 bridges has the highest number of long span bridges followed by Pennsylvania (241) and New York (234). Nevada with 5 bridges has the fewest number in the nation.

Southeast - Among the seven southern states (*Alabama, Florida, Georgia, Louisiana, Mississippi, North Carolina and South Carolina*) Alabama has the highest number of reinforced concrete bridges (13), followed by Mississippi (4) and Florida (3). The largest number of steel bridges are in Louisiana (113), followed by Alabama (82) and Florida (81). The largest number of prestressed concrete bridges are in Georgia (28) followed by Florida (10) and Louisiana (5). North Carolina (20) has the fewest number of bridges with Louisiana (113), the highest number.

Table 2.6 State-Wise Breakdown of RC, Steel and PSC Bridges.

State	Reinforced Concrete		Steel		Prestressed Concrete		Total
	#	%	#	%	#	%	
Alabama	13	13.5	82	85.4	1	1.0	96
Alaska	0	0.0	40	95.2	2	4.8	42
Arizona	1	2.9	20	58.8	13	38.2	34
Arkansas	3	5.9	48	94.1	0	0.0	51
California	37	11.8	100	31.9	175	56.2	312
Colorado	1	2.0	33	67.3	15	30.6	49
Connecticut	0	0.0	56	88.9	7	11.1	63
Delaware	2	15.4	11	84.6	0	0.0	13
D.C.	3	20.0	12	80.0	0	0.0	15
Florida	3	3.2	81	86.2	10	10.6	94
Georgia	1	1.6	34	54.0	28	44.4	63
Hawaii	1	10.0	2	20.0	7	70.0	10
Idaho	1	2.4	40	95.2	1	2.4	42
Illinois	5	2.8	173	96.1	2	1.1	180
Indiana	0	0.0	26	86.7	4	13.3	30
Iowa	2	4.5	42	95.5	0	0.0	44
Kansas	26	28.6	65	71.4	0	0.0	91
Kentucky	3	2.3	123	94.6	4	3.1	130
Louisiana	1	0.9	106	93.8	6	5.3	113
Maine	51	50.5	50	49.5	0	0.0	101
Maryland	1	1.8	56	98.2	0	0.0	57
Massachusetts	1	2.4	40	97.6	0	0.0	41
Michigan	0	0.0	27	79.4	7	20.6	34
Minnesota	8	11.6	58	84.1	3	4.3	69
Mississippi	4	6.6	53	86.9	4	6.6	61
Missouri	2	2.3	84	97.7	0	0.0	86
Montana	0	0.0	48	100.0	0	0.0	48

Table 2.6 (Contd.) State-Wise Breakdown of RC, Steel and PSC Bridges.

State	Reinforced Concrete		Steel		Pre-stressed Concrete		Total
	#	%	#	%	#	%	
Nebraska	0	0.0	25	96.2	1	3.8	26
Nevada	0	0.0	2	40.0	3	60.0	5
New Hampshire	0	0.0	26	100.0	0	0.0	26
New Jersey	3	7.0	40	93.0	0	0.0	43
New Mexico	0	0.0	9	100.0	0	0.0	9
New York	6	2.6	221	94.4	7	3.0	234
North Carolina	2	10.0	17	85.0	1	5.0	20
North Dakota	0	0.0	16	94.1	1	5.9	17
Ohio	5	4.5	104	93.7	2	1.8	111
Oklahoma	2	2.2	91	97.8	0	0.0	93
Oregon	15	14.2	75	70.8	16	15.1	106
Pennsylvania	15	6.2	224	92.9	2	0.8	241
Rhode Island	0	0.0	8	100.0	0	0.0	8
South Carolina	3	7.5	35	87.5	2	5.0	40
South Dakota	0	0.0	11	100.0	0	0.0	11
Tennessee	6	4.3	130	92.2	5	3.5	141
Texas	4	2.4	151	91.5	10	6.1	165
Utah	0	0.0	41	100.0	0	0.0	41
Vermont	0	0.0	35	100.0	0	0.0	35
Virginia	3	4.6	61	93.8	1	1.5	65
Washington	20	9.0	181	81.2	22	9.9	223
West Virginia	3	9.1	30	90.9	0	0.0	33
Wisconsin	0	0.0	56	98.2	1	1.8	57
Wyoming	0	0.0	14	100.0	0	0.0	14
Puerto Rico	2	25.0	4	50.0	2	25.0	8

2.7 Distribution by Type

NBI recognizes 23 types of bridge decks (see Table 2.2). The analysis presented in this section attempted to determine the relationship between material and deck types. The results are summarized in Table 2.7. The following observations may be made:

Reinforced Concrete - The two most widely used bridge types are arch deck (82) and Tee beam (58). In percentage terms, these numbers reflect 31.7% (arch deck), 22.4% (Tee beam). Eight types allowed by NBI, are not used for reinforced concrete bridge construction. These are orthotropic, suspension, stayed girder, movable lift, movable bascule, movable swing, tunnel and mixed types.

Steel - The two most widely used bridge types are stringer multi-beam or girder (1154) and truss-thru (950). In percentage terms, these numbers reflect 37% (girder) and 30.5% (truss-thru). Four types allowed by NBI are not used. These are tee beam, tunnel, culvert and channel beam.

Prestressed Concrete - Box girder type bridges are the most widely used and make up a total of 247 of the 365 prestressed bridges. This constitutes almost 68% of the total number of prestressed bridges. The other two types used with some frequency are box beam girders (66) and multi-beam girders (46). Only seven of the 23 types recognized by NBI are used in prestressed concrete construction. In addition to the eight types not used in reinforced concrete construction, eight others are also not used. These are girder and floorbeam systems, frame, truss-deck, truss-thru, arch-deck, arch-thru, channel beam and other.

Table 2.7 Relationship Between Bridge and Material Types.

Type	Concrete		Steel		Prestressed Concrete	
	#	%	#	%	#	%
Slab	17	6.6	1	0.0	3	0.8
Stringer Multi-beam or Girder	11	4.2	1154	37.0	46	12.8
Girder and Floorbeam System	2	0.8	327	10.5	0	0.0
Tee Beam	58	22.4	0	0.0	2	0.5
Box Beam or Girders – Multiple	27	10.4	53	1.7	221	60.4
Box Beam or Girders – Single or Spread	13	5.0	68	2.2	66	18.0
Frame	2	0.8	10	0.3	0	0.0
Orthotropic	0	0.0	6	0.2	0	0.0
Truss – Deck	1	0.4	189	6.1	0	0.0
Truss – Thru	1	0.4	950	30.5	0	0.0
Arch – Deck	82	31.7	47	1.5	0	0.0
Arch – Thru	6	2.3	76	2.4	0	0.0
Suspension	0	0.0	28	0.9	0	0.0
Stayed Girder	0	0.0	2	0.1	0	0.0
Movable Lift	0	0.0	55	1.8	0	0.0
Movable Bascule	0	0.0	84	2.7	0	0.0
Movable Swing	0	0.0	55	1.8	1	0.3
Tunnel	0	0.0	0	0.0	0	0.0
Culvert	4	1.5	0	0.0	0	0.0
Mixed Types	0	0.0	2	0.1	0	0.0
Segmental Box Girder	7	2.7	4	0.1	26	7.1
Channel Beam	1	0.4	0	0.0	0	0.0
Other	27	10.4	6	0.2	0	0.0
Total	259	100.0	3117	100.0	365	100.0

2.8 Distribution by Time

To help identify underlying trends, bridge construction is examined over 5-year intervals starting from 1920. This information is summarized in Table 2.8. The first prestressed concrete bridge in the 200-600 ft range was constructed between 1956-1960. Since then, steel's share has fallen steadily from 94.1% to 69.8% in most recent period between 1991-95. A year by year breakdown from 1960-96 is given in Tables 2.9-2.12.

Table 2.8 Bridge Construction - 5 Year Periods.

Year	Reinforced Concrete		Steel		Prestressed Concrete		Total
	Number	%	Number	%	Number	%	
Pre-1920	13	5.4	227	94.6	0	0.0	240
1921-25	11	13.3	72	86.7	0	0.0	83
1926-30	31	16.9	152	83.1	0	0.0	183
1931-35	23	10.8	189	89.2	0	0.0	212
1936-40	15	6.8	206	93.2	0	0.0	221
1941-45	4	6.0	63	94.0	0	0.0	67
1946-50	8	5.3	143	94.7	0	0.0	151
1951-55	54	24.0	171	76.0	0	0.0	225
1956-60	12	5.1	222	94.1	2	0.8	236
1961-65	14	5.6	231	92.8	4	1.6	249
1966-70	16	5.6	256	89.5	14	4.9	286
1971-75	14	4.3	253	76.9	62	18.8	329
1976-80	7	2.7	199	78.0	49	19.2	255
1981-85	12	4.3	208	74.6	59	21.1	279
1986-90	14	3.6	295	75.1	84	21.4	393
1991-95	11	3.4	229	69.8	88	26.8	328
1996	0	0.0	1	25.0	3	75.0	4
Total	259	6.9	3117	83.3	365	9.8	3741

Table 2.9 Bridge Construction - 1960 to 1969.

Year	Reinforced Concrete		Steel		Prestressed Concrete		Total
	#	%	#	%	#	%	
1960	3	6.4	44	93.6	0	0	47
1961	3	8.1	35	92.1	0	0	38
1962	5	10.0	43	86.0	2	4.0	50
1963	4	6.8	53	89.8	2	3.4	59
1964	1	2.2	45	97.8	0	0	46
1965	1	1.8	55	96.5	1	0	57
1966	2	4.2	45	93.8	1	2.1	48
1967	2	3.6	52	94.5	1	1.8	55
1968	4	7.7	48	92.3	0	0	52
1969	4	6.3	56	88.9	3	4.8	63
Total	29	5.6	476	92.4	10	1.9	515

Table 2.10 Bridge Construction - 1970 to 1979.

Year	Reinforced Concrete		Steel		Prestressed Concrete		Total
	#	%	#	%	#	%	
1970	4	5.9	55	80.9	9	5.9	68
1971	2	2.6	59	77.6	15	2.6	76
1972	1	1.9	41	75.9	12	1.7	54
1973	6	9.1	54	81.8	6	6.2	66
1974	0	0.0	55	83.3	11	0	66
1975	5	7.5	44	65.7	18	57.5	67
1976	1	1.5	53	79.1	13	1.5	67
1977	2	4.2	33	68.8	13	4.4	48
1978	2	4.1	34	69.4	13	4	49
1979	1	2.6	33	86.8	4	2	38
Total	24	4.0	461	77.0	114	19.0	599

Table 2.11 Bridge Construction - 1980 to 1989.

	Reinforced Concrete		Steel		Prestressed Concrete		Total
	#	%	#	%	#	%	
1980	1	1.6	45	72.6	16	25.8	62
1981	5	7.2	58	84.1	6	8.7	69
1982	2	3.8	37	69.8	14	26.4	53
1983	3	6.4	34	72.3	10	21.3	47
1984	1	1.5	49	74.2	16	24.2	66
1985	1	2.2	30	66.7	14	31.1	45
1986	2	3.2	46	74.2	14	22.6	62
1987	4	5.5	57	78.1	12	16.4	73
1988	4	4.7	61	71.8	20	23.5	85
1989	2	2.4	57	69.5	23	28.0	82
Total	25	3.9	474	73.6	145	22.5	644

Table 2.12 Bridge Construction - 1990 to 1996.

	Reinforced Concrete		Steel		Prestressed Concrete		Total
	#	%	#	%	#	%	
1990	2	2.2	74	79.6	17	18.3	93
1991	4	4.7	65	76.5	16	18.8	85
1992	3	4.9	47	77.0	11	18.0	61
1993	1	1.3	46	61.3	28	37.3	75
1994	2	2.9	48	69.6	19	27.5	69
1995	1	2.1	33	68.8	14	29.2	48
1996	0	0.0	1	25.0	3	75.0	4
Total	13	3.0	314	72.2	108	24.8	435

2.9 Florida - Overview

The analysis of Florida's bridges is based on the information provided by the Florida Department of Transportation. The format of the results contained in Sections 2.9.1-2.9.4, is very similar to that for the national survey presented in Sections 2.4-2.8.

Table 2.11 shows the distribution of reinforced concrete, steel and prestressed concrete bridges in the state of Florida as of Dec 31st 1997. Of the 106 bridges (*compared to only 94 in Table 2.6*) , about 83% are made of steel, 14% of prestressed concrete and 3% of reinforced concrete. While the percentage of steel bridges is nearly identical to the national average, the proportion of prestressed concrete bridges is somewhat higher (see Table 2.3).

Table 2.12 summarizes information on the average age of Florida's bridges. The first longer span reinforced concrete and prestressed concrete bridges were only built in the 1980's. Not surprisingly, steel bridges are older. This contrasts with the national picture where average ages tend to be significantly higher with reinforced concrete bridges being the oldest (see Table 2.4).

Table 2.11 Breakdown by Material Type.

Type	Number	Percentage
Concrete	3	2.8
Steel	88	83.0
Prestressed Concrete	15	14.2
Total	106	100

Table 2.12 Average Age by Material Type.

Type	Period	Number	Average Age (years)
Concrete	1984-95	3	8
Steel	1927-97	88	14
Prestressed	1984-97	15	6

2.9.1 Distribution by Span

This section provides the same information for Florida as Section 2.5 in the national survey. It reviews data on Florida bridges spanning between 200-600 ft. span and links this to the type of material used in their construction. As before, the span range is examined for 50 ft intervals, e.g. 300-350 ft. For each such range, both the number of bridges and their percentage for the total number constructed are included. In addition cumulative values both in terms of numbers and percentages are also provided. Table 2.13 provides a summary of the results.

Inspection of Table 2.13 shows that nearly 70% of all bridges between 200-600 ft have spans less than or equal to 250 ft. This is a larger proportion than the 60% value nationally (see Table 2.5). Over 97% of Florida's bridges have spans less than 400 ft compared to the 90% figure nationally. Finally, there are no bridges that span above 550 ft. Nationally, 1.4% of the bridges span this range.

The results of the analysis provide added confirmation of the findings from the national survey. Structures spanning above 400 ft tend to be one-off structures that may not be amenable to generic solutions.

Table 2.13 Distribution by Span and Material Type.

Span	RC		Steel		PC		Total	
ft	#	Cumulative	#	Cumulative	#	Cumulative	#	Cumulative
200-250	2 (66.6%)	2 (66.7%)	60 (68.2%)	60 (68.2%)	12 (80.0%)	12 (80.0%)	74 (69.8%)	74 (69.8%)
251-300	1 (33.3%)	3 (100%)	21 (23.9%)	81 (92.0%)	1 (6.7%)	13 (86.7%)	23 (21.7%)	97 (91.5%)
301-350	0	3 (100%)	4 (4.5%)	85 (98.9%)	0 (0.0%)	13 (86.7%)	4 (3.8%)	100 (94.3%)
351-400	0	3 (100%)	2 (2.3%)	87 (100.0%)	0 (0.0%)	13 (86.7%)	2 (1.9%)	103 (97.2%)
401-450	0	3 (100%)	1 (1.1%)	88 (100.0%)	0 (0.0%)	13 (86.7%)	1 (0.9%)	104 (98.1%)
451-500	0	3 (100%)	0	88 (100.0%)	0 (0.0%)	13 (86.7%)	0 (0.0%)	104 (98.1%)
501-550	0	3 (100%)	0	88 (100.0%)	2 (0.0%)	15 (86.7%)	2 (1.9%)	106 (100.0%)
551-600	0	3 (100%)	0	88 (100.0%)	0	15 (86.7%)	0	106 (100.0%)

2.9.2 Distribution by Type

This section presents information on the different types of bridge decks made of reinforced concrete, steel and prestressed concrete that have been used in long span construction in Florida. Table 2.14 summarizes the results. The following observations may be made:

Reinforced Concrete - Only three bridges have been built using reinforced concrete. Each one is of a different type.

Steel - The two most widely used bridge types are multi-beam (53) and box type (23). In percentage terms, these represent 60% (multi-beam) and 26% (box) of all steel bridges. Nation-wide, multi-beam decks are also the most popular but truss-thru bridges rank second (see Table 2.7).

Prestressed Concrete - Segmental box girders make up 60% of all the long span prestressed bridges built in the state. Multi-beam girders make the remaining 40%. Nationwide, box girder type bridges make up nearly 68% of the total.

Table 2.14 Relationship Between Bridge and Material Types.

Type	Concrete		Steel		Prestressed Concrete	
	#	%	#	%	#	%
Slab	1	33.3	0	0.0	0	0.0
Stringer Multi-beam or Girder	1	33.3	53	60.2	6	40.0
Girder and Floorbeam System	0	0.0	4	4.5	0	0.0
Box Beam/Girders Single/Spread	1	33.3	23	26.1	0	0.0
Truss – Thru	0	0.0	2	2.3	0	0.0
Arch – Thru	0	0.0	1	1.1	0	0.0
Suspension	0	0.0	1	1.1	0	0.0
Movable Lift	0	0.0	1	1.1	0	0.0
Movable Bascule	0	0.0	1	1.1	0	0.0
Movable Swing	0	0.0	2	2.3	0	0.0
Seg. Box Girder	0	0.0	0	0.0	9	60.0
Total	3	100.0	88	100.0	15	100.0

2.9.3 Year-Wise Breakdown

To identify if there are any underlying trends, bridge construction is examined over 5-year intervals starting from 1935. This information is summarized in Table 2.15.

Prior to 1976, relatively few longer span bridges were constructed in the state. In the two succeeding decades, a total of eight one bridges were constructed, with the lion's share of 62 bridges being constructed from 1986-95. Whereas steel was exclusively used for bridge construction until 1980, more concrete has been used since the 1980's. In the five year period ending 1995, 82.3% of the bridges were made of steel. This is higher than the corresponding period from the national survey where it was only 69.8% (Table 2.8).

For completeness, a year-by-year breakdown of bridge construction from 1960-97 is given in Table 2.16.

Table 2.15 Bridge Construction - 5 Year Periods.

Year	Reinforced Concrete		Steel		Prestressed Concrete		Total
	#	%	#	%	#	%	
1935-40	0	0	1	100	0	0	1
1941-45	0	0	1	100	0	0	1
1946-50	0	0	2	100	0	0	2
1951-55	0	0	2	100	0	0	2
1956-60	0	0	1	100	0	0	1
1961-65	0	0	2	100	0	0	2
1966-70	0	0	5	100	0	0	5
1971-75	0	0	1	100	0	0	1
1976-80	0	0	8	100	0	0	8
1981-85	1	8.3	10	83.3	1	8.3	12
1986-90	1	3.7	21	77.7	5	18.5	27
1991-95	1	2.9	29	82.3	5	14.3	35
1996-97	0	0	4	44.4	4	44.4	8
Total	3	2.9	87	82.3	15	14.3	105

Table 2.16 Year-by Year Bridge Construction - 1960 to 1997.

Year	Reinforced Concrete		Steel		Pre-stressed Concrete		Total
1960	0	0	1	100	0	0	1
1964	0	0	1	100	0	0	1
1965	0	0	1	100	0	0	1
1967	0	0	1	100	0	0	1
1968	0	0	1	100	0	0	1
1969	0	0	1	100	0	0	1
1970	0	0	2	100	0	0	2
1973	0	0	1	100	0	0	1
1976	0	0	2	100	0	0	2
1978	0	0	5	100	0	0	5
1980	0	0	1	100	0	0	1
1981	0	0	4	100	0	0	4
1982	0	0	2	100	0	0	2
1984	1	20.0	3	60	1	20.0	5
1985	0	0	1	100	0	0	1
1986	0	0	0	0	2	100	2
1987	0	0	3	100	0	0	3
1988	0	0	7	77.7	2	22.3	9
1989	1	20.0	3	60	1	20.0	5
1990	0	0	8	100	0	0	8
1991	0	0	5	83.3	1	16.7	6
1992	0	0	0	0	1	100	1
1993	0	0	9	81.8	2	18.2	11
1994	0	0	8	88.8	1	11.2	9
1995	1	12.5	7	87.5	0	0	8
1996	0	0	1	33.3	2	66.7	3
1997	0	0	3	60.0	2	40.0	5
Total	3	3.1	81	82.3	15	15.3	99

2.9.4 Bridges Crossing Navigable Channels

Information contained in the data received from FDOT, allowed identification of bridges over navigable channels. Of the 106 bridges, 28 were constructed over navigable channels. About a sixth (5) of these were made of concrete with the remaining 23 made of steel. Complete details are summarized in Table 2.17.

Table 2.17 Navigable Fixed Bridges.

Material	Structure Type	Year	Span (ft)	Vertical Clearance (ft)
Concrete	Stringer Beam/Girder	1997	235	43
Steel	Truss Thru	1938	281	52
Steel	Suspension	1947	421	20
Steel	Truss Thru	1960	328	50
Steel	Stringer Beam/Girder	1964	201	49
Steel	Stringer Beam/Girder	1965	251	50
Steel	Stringer Beam/Girder	1967	201	75
Steel	Stringer Beam/Girder	1969	201	75
Steel	Stringer Beam/Girder	1970	251	65
Steel	Stringer Beam/Girder	1970	251	65
Steel	Girder & Floorbeam	1978	206	65
Steel	Girder & Floorbeam	1978	206	65
Steel	Girder & Floorbeam	1978	293	35
Steel	Girder & Floorbeam	1978	293	35
Steel	Stringer Beam/Girder	1987	301	56
Steel	Stringer Beam/Girder	1988	301	56
Steel	Stringer Beam/Girder	1988	301	56
Steel	Stringer Beam/Girder	1988	201	64
Steel	Stringer Beam/Girder	1988	291	65
Steel	Stringer Beam/Girder	1990	211	65
Steel	Stringer Beam/Girder	1991	261	65
Steel	Stringer Beam/Girder	1993	221	67
Steel	Stringer Beam/Girder	1997	221	43
Steel	Stringer Beam/Girder	1997	221	43
Prestressed	Stringer Beam/Girder	1994	201	45
Prestressed	Stringer Beam/Girder	1991	201	65
Prestressed	Stringer Beam/Girder	1993	251	75
Prestressed	Segmental Box Girder	1993	226	135

2.10 Conclusions

Based on the information presented in the chapter, the following conclusions may be drawn:

1. The vast majority of bridges in the 200-600 ft span have spans less than 400 ft. The data analyzed indicates that 89.8% of the nation's bridges have spans between 200-400 ft. (Table 2.5). In Florida, this number is *above 97%* (see Table 2.13). This suggests that the focus of a generic solution should be limited to the 200-400 ft range. Spans above 400 ft tend to be unique structures where generic solutions may not be appropriate.

2. The share of long span steel bridges declined nationwide from 94.1% in the 5-year period 1956-60 to 69.8% in the period most recently concluded 5-year period between 1991-95 (Table 2.8). The corresponding percentages for Florida are 100% and 82.3% respectively (Table 2.15). Thus, while steel's share of the market for long span bridges in Florida has declined, its rate of decline is much lower.

3. Florida's long span bridges are all relatively new; the oldest bridges made of steel average 14 years while the youngest, made of prestressed concrete are only 6 years old (see Table 2.12). The newness of the bridges makes it more problematic to identify deficiencies in existing design.

4. The most commonly used bridge types nationally for spanning 200-600 ft are *arch deck* (RC), *multi-beam girder* (Steel) and *box girder* (PC) (Table 2.7). This is also true for Florida with the exception that reinforced concrete is rarely used in the state. Only three reinforced concrete bridges have been constructed, each being a different type (Table 2.14).

References

2.1 Office of Engineering Bridge Division (1988). "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nations's Bridges," Report No. FHWA-ED-89-044, U.S. Department of Transportation, Washington, DC.

3. HIGH PERFORMANCE CONCRETE

3.1 Introduction

High strength concrete (HSC) and high performance concrete (HPC) are terms that have sometimes been used interchangeably. HSC was defined as having a minimum compressive strength of 6,000 psi [41 MPa] in 1984 [3.1]. HPC, however, refers to specially engineered concrete that meets additional requirements; for example, the Federal Highway Administration (FHWA) defines HPC in terms of long-term performance criteria that consists of four durability and four strength parameters [3.2].

This chapter provides a brief overview of HPC. Section 3.2 discusses criteria used by FHWA in defining HPC. Section 3.3 outlines some advantages of using HPC while Section 3.4 focuses on field applications and costs. Finally, Section 3.5 addresses some challenges that may need to be overcome.

3.2 FHWA Criteria

FHWA defines high performance concrete in terms of eight performance criteria consisting of four *durability* and four *strength* criteria. The eight parameters were the result of an investigation of general field conditions that cause concrete to deteriorate including climate, exposure effects and loads. It is intended that these definition parameters will be updated regularly in line with improvements in technology and field experience.

Durability criteria comprise freeze-thaw, scaling, abrasion and chloride penetration. Strength criteria comprise of compressive strength, modulus of elasticity and creep/shrinkage characteristics. Test methods for evaluating these performance criteria have been specified (see Table 3.1). These have to be performed on concrete samples moist or submersion cured for 56 days [3.2].

Table 3.1 defines four grades that are identified by the numerals 1-4. The compressive strength of the concrete increases from 6-8 ksi (41-55 MPa) for Grade 1 to over 14 ksi (97 MPa) for Grade 4. The corresponding elastic modulus varies from 4-6 msi (28-40 GPa) to over 7.5 msi (50 GPa). To specify a HPC concrete mixture, a user specifies the level of performance required for each of the states. Thus, a concrete may need to perform at Grade 1 in strength and elasticity, Grade 3 in shrinkage and scaling resistance and Grade 2 in all other categories.

Table 3.1 FHWA Performance Characteristics for HPC [3.2].

Performance Characteristics	**Test Method**	**FHWA HPC Performance Grade**			
		1	**2**	**3**	**4**
Freeze-thaw durability (dynamic modulus after 300 cycles)	AASHTO T 161 ASTM C666 Proc. A	60-80%	≥80%		
Scaling resistance x = visual surface rating after 50 cycles	ASTM C 672	x = 4,5	x = 2,3	x = 0,1	
Abrasion resistance x=average depth of wear in mm	ASTM C 944	1-2	0.5-1	< 0.5	
Chloride penetration (coulombs)	AASHTO T 277 ASTM C 1202	2000-3000	800-2000	≤ 800	
Compressive Strength ksi (MPa)	AASHTO T 2 ASTM C 39	6-8 (41-55)	8-10 (55-69)	10-14 (69-97)	≥ 14 (≥ 97)
Modulus msi (GPa)	ASTM C 469	4-6 (28-40)	6-7.5 (40-50)	>7.5 (>50)	
Shrinkage ($\mu\epsilon$)	ASTM C 157	600-800	400-600	<400	
Creep ($\mu\epsilon$/psi (MPa))	ASTM C 512	0.41-0.52 (60-75)	0.31-0.41 (45-60)	0.31-0.21 (30-45)	< 0.21 (< 30)

In addition to the performance criteria, recommended performance for different exposures have also been provided. These are summarized in Table 3.2.

Table 3.2 FHWA Recommendations for HPC Grades [3.2].

Exposure Condition	**Recommended HPC Grade for Exposure Condition**				
	N/A	**1**	**2**	**3**	**4**
Freeze-Thaw (cycles/year)	< 3	3-50	>50		
Scaling Resistance (applied salt - tons/lane-mile-year)	< 5	> 5			
Abrasion Resistance (average daily traffic, studded tires allowed)	no studs no chains	< 50,000	50,000-100,000	>100,000	
Chloride Penetration (applied salt - tons/lane-mile-year)	< 1	1-3	3-6	> 6	

3.3 Advantages

The use of high performance concrete with its improved long-term durability, is anticipated to provide significant benefits in the construction of bridges. The potential for longer service life with reduced maintenance requirements at a time of increasing infrastructure age and decreasing budgets makes it very attractive.

The higher strengths that are possible with HPC, when used in the design of beams, for example, can result in lower or comparable initial costs for HPC relative to normal concrete bridges. This is due to the potential for longer spans, fewer units to fabricate, transport, and erect, even though the unit cost of HPC may be higher. Its higher initial cost is of primary concern in bidding. However, its anticipated life is between 75 to 100 years, possibly double a bridge's service life.

At the moment, the economic evaluations of HPC based on life-cycle costs are not readily available. As a relatively new concept, the economic benefits related to improved long-term durability of HPC bridges are not quantifiable at this time. An economic evaluation will therefore be based on cost comparisons that can be made at this early stage of HPC implementation.

3.4 Applications

European countries have been routinely using HPC for nearly 10 years. More than 100 bridges have also been built in Canada including the 8-mile long Northumberland Strait Crossing connecting Prince Edward Island and New Brunswick [3.3].

The Virginia Department of Transportation has used HPC on 13 bridge projects following completion of two demonstration structures. Other states that have used HPC include Alabama, Colorado, Delaware, Georgia, Louisiana, Nebraska, New Hampshire, New Mexico, New York, North Carolina, Ohio, South Dakota and Washington [3.3].

The Louetta Road overpass on State Highway 249 near Houston is the first bridge in the United States to fully utilize HPC in all aspects of design and construction. Texas DOT's second HPC bridge located near San Angelo carries the eastbound lanes of US Route 67 over the North Concho River, US 87 and the South Orient railroad. The 291-m (950 ft) HPC bridge runs parallel to a 292 m bridge built of normal concrete. The first spans of the two bridges are identical and therefore provide an ideal basis for comparing cost and performance of the two bridges.

A benefit of HPC was clearly evident in the first span for this bridge. **Seven** normal concrete deep AASHTO Type IV beams were required compared to **four** high-strength HPC beams over the same length. This reduction resulted in a 43 percent decrease in the number of beams fabricated, transported, and erected. Additionally, it resulted in a reduction in the number of substructures [3.4].

Table 3.3 provides a summary of the design compressive strengths for components in the two bridges. The only component in the westbound lane that was HPC was the cast-in-place deck that was designed to provide added durability.

Table 3.3 Concrete Strength Comparisons for San Angelo Overpass [3.4].

Component	**HPC bridge (eastbound)**	**Normal concrete (westbound)**
Prestressed I beam	5,800 - 14,700 psi	5,000-8,900 psi
Cast-in-place deck	6,000 psi	4,000 psi (HPC)
Prestressed deck panels	6,000 psi	5,000 psi
Substructure : Caps	8,000 psi	6,000 psi
Substructure: Columns	6,000 psi	3,600 psi

The HPC bridge cost approximately \$47 per sq. ft versus \$41 for the conventional bridge [3.3], approximately 15% higher. A similar comparison for the Louetta Bridge indicated a 2% difference in as-bid costs [3.4].

3.5 Challenges

It is important to recognize that higher prestressing forces are required for fabricating HPC specimens because of their higher strength. It has been speculated that the advantages of going beyond 10,000 psi may be limited because of difficulties in achieving the requisite prestressing forces. For the San Angelo project, 0.6 in. strands were used which needed the prestressing bed to be modified. Moreover, because of a FHWA moratorium on strands larger than 0.5 in. additional testing had to be formed. Transportation and handling of the 157 ft beams used in the San Angelo bridge also project posed problems.

There has been an increase in demand for improved education of workers in the concrete industry. High Performance Concrete requires substantial specialized skills and should not just be made available for use to anyone without some form of training or certification requirement, as is the case in the steel industry. However, with most of the initial problems surmounted, and increase in availability of expertise with time, it is expected that production costs would drastically reduce and as such future HPC projects will be more cost-competitive.

References

3.1 Farny, J. A., and Panarese, W. C., (1994) "High-Strength Concrete," Portland Cement Assoc., Skokie, Ill.

3.2 Goodspeed, C., Vanikar, S. and Cook, R. (1996). "HPC Defined for Highway Structures", Concrete International, Vol. 18, No 2, pp. 62-67.

3.3 ENR (1998). "From Buildings to Bridges", January 19, pp. 40-44 .

3.4 Ralla, M.L. (1998). "Texas High Performance Bridges - How Much Do They Cost", Concrete International, Vol. 20, No. 3, pp. 71-74.

4. HIGH PERFORMANCE STEEL

4.1 Introduction

Structural steel commonly used in bridge construction has a yield strength of around 50 ksi (345 MPa). Available 70 ksi (485 MPa) and 100 ksi (690 MPa) higher strength steels offer significant economic advantages but their use has been limited because they are more difficult to weld. This is because 'weldability' is inversely proportional to higher carbon content (or carbon equivalent) that is used to achieve the higher strength.

High performance steel (HPS) refers to a newly developed grade of higher strength, low carbon steel that combines the strength, ductility, corrosion resistance and formability in lower strength steels with those for weldability and toughness. HPS was developed under a cooperative research program between the Federal Highway Administration (FHWA), the U.S. Navy and the American Iron and Steel Institute (AISI) and became commercially available in less than two and a half years from project initiation. In 1997, AISI, FHWA and the US Navy were recipients of the Charles Pankow Award for Innovative Applications. This new commercially available weathering steel, HPS-485W, satisfies the requirements of A709-485W so that it can be ordered under existing specifications [4.1].

This chapter provides a brief overview of HPS with Section 4.2 containing background information on its development, testing and evaluation. Section 4.3 provides information on demonstration bridges. Additional information on these bridges may be found in Chapter 10.

4.2 Background

The major goal in the development of HPS was to improve the weldability of high strength steel. This was achieved by varying processing methods and alloy composition until an optimum combination was reached. This was found to be a modified version of existing A709 grade 485W quenched and tempered steel.

Table 4.1 compares the differences in the composition of HPS-485W and A709-485W. The major difference is the almost 50% reduction in carbon content from 0.19% to 0.10%. Strength was maintained through processing changes, micro-alloy additions and alloy adjustments.

A variety of tests are being conducted to evaluate the weldability and structural performance of steel members fabricated using HPS-485W. Preliminary results indicate that

Table 4.1 Comparison of Composition of HPS-70 and Conventional Steels [4.1].

Comparison of HPS-70 to Conventional Steels											
Steel	**C**	**Mn**	**P**	**S**	**Si**	**Cu**	**Ni**	**Cr**	**Mo**	**V**	**Al**
A709-70W	0.19 max	0.8 to 1.35	0.035 max	0.04 max	0.02 to 0.65	0.2 to 0.4	0.5 max	0.4 to 0.7		0.02 to 0.1	0
HPS-70W	0.08 to 0.11	1.15 to 1.3	0.02 max	0.06 max	0.35 to 0.45	0.29 to 0.38	0.28 to 0.38	0.5 to 0.6	0.04 to 0.08	0.05 to 0.07	0.01 to 0.04

HPS-485W is capable of being welded without preheat and interpass control using both submerged arc welding (SAW) and shielded metal arc welding (SMAW) processes [4.1]. Structural performance is being evaluated through full-scale fatigue and fracture tests being conducted at the FHWA Structures Laboratory in McLean, VA.

4.3 Demonstration Bridges

FHWA is providing funds for research, documentation, quality control, and field monitoring of HPS structures. States provide the funding for any additional design or construction cost due to use of HPS. Two demonstration bridges have already been constructed in Nebraska and Tennessee. Description of both these structures may be found in Chapter 10. Both states have begun their second HPS projects. Several other states are also planning to use it. New York State Thruway Authority has seven HPS projects in the works. Massachusetts, North Carolina, Pennsylvania and Virginia also have HPS projects in the pipeline [4.2, 4.3]. Michigan and West Virginia have also expressed interest in developing future projects [4.1].

Nebraska Department of Roads completed the first HPS bridge in the US last year. This cost 20% more per square foot than a conventional span on the same project as the intent was primarily to see "if the steel could be fabricated" [4.2].

The second HPS bridge completed by Tennessee DOT is the first to use the new AASHTO LRFD specifications. Cost estimates indicate that the steel weight was reduced by 25% resulting in a 11% overall reduction in the total cost of fabricating and erecting this bridge [4.3]. Thus, HPS leads to a reduction in initial construction costs.

Limited experience in construction with HPS has given initial cost savings in materials and fabrication due to the reduced weight of steel. Additional savings will be possible as the welding and fabrication procedures are enhanced and new bridge designs are developed to take advantage of the higher strength steels.

Those who have been involved in HPS projects have been pleased with the improved attributes provided by HPS. As designers develop confidence in the ability to weld HPS steels and as the cost savings are realized in completed structures the use of HPS will increase.

The growth of HPS will require modifications to plants to heat and cool longer lengths of steel. Currently, only 50 ft lengths can be produced so that HPS girders will require more splices until larger ovens can be installed [4.2]. The use of higher strength will lead to lighter and more slender shapes. Local and overall instability may prevent HPS members from achieving its yield strength or significantly reduce inelastic rotation capacity after yielding. This may lead to the acceptance of innovative shapes such as those proposed by J. Muller International's Chicago office for AISI [4.4].

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5. ALUMINUM

5.1 Introduction.

Aluminum is a lighweight, high-strength, ductile, corrosion-resistant metal with excellent fatigue characteristics. It is a truly 100% recycable material that is weldable, extrudable into desired shapes with excellent longetivity and durability. The light weight (*aluminum's density is approximately one-third that of steel*) is particularly advantageous for long span structures where dead load constitutes a significant proportion of the loading. Light weight is also beneficial in seismic areas because it results in lower inertial forces. In either application, it leads to lighter foundations, and reduced installation and erection costs.

The use of aluminum in bridge construction is not new. In 1934, the historic Smithfield Street lenticular truss bridge in Pittsburgh installed an aluminum bridge deck and support structure to reduce dead load and improve load-carrying capacity. More recently, developments in the US and overseas have led to an increase in the use of aluminum. The most ambitious aluminum structure being contemplated is the five span 300 m (985 ft) highway bridge across the Smedasundet in Risøy in Norway [5.1]. In the United States, focus has been mainly on deck replacement where aluminum is beginning to make a comeback [5.2]. This chapter provides a brief overview of aluminum used in bridge construction. Section 5.2 focuses on deck systems developed in the United States. Section 5.3 presents information on aluminum bridge construction overseas.

5.2 Deck Systems

Aluminum's light weight and durability makes it especially suitable in deck replacement where it combines rapid installation with increased load carrying capacity.

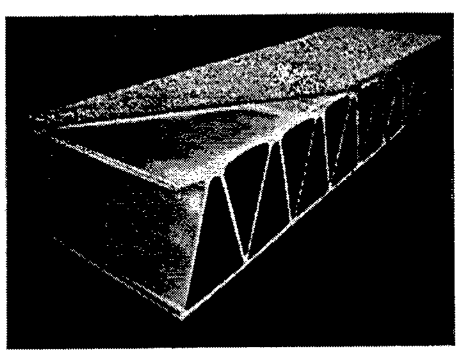

Figure 5.1 Alumadeck Bridge System [5.4].

Though aluminum is more costly than conventional decking materials, its life-cycle may be lower [5.1]. According to a study conducted by Alcoa in the 1980's and reported in Ref. 5.3, "*the average downtime for a bridge receiving an aluminum deck was 24 days........compared to 10-12 months for a comparable job using concrete*".

High Steel Structures, Inc. and Reynolds Metals Company have formed an alliance to develop an innovative aluminum bridge system, Alumadeck, that is geared for deck replacement applications [5.4]. Alumadeck uses hollow aluminum extrusions typically 12 in. (30 mm) wide and 8 in. (20 mm) deep (Fig. 5.1). The top side of the deck panel is covered with a shop-applied wearing surface to ensure a durable and skid-resistant riding surface. The individual aluminum extrusions are welded to form the prefabricated deck panel that is delivered to the installation site. Shipping considerations dictate panel lengths and widths.

For longitudinal girder bridges, the panel width matches the girder spacing. Shear studs on the girder top flange slip through holes in the bottom flange of the deck. The deck panels are bolted and high strength grout is used to fill the void between the panels. The aluminum deck acts much like a concrete slab transferring loads in the longitudinal and transverse directions though it has significant negative moment capacity.

5.2.1 Advantages

- **Combination of light weight and high structural and fatigue strength:** an 8 in. deep aluminum deck weights approximately 25 lb/ft^2 comparing to 100 lb/ft^2 for conventional concrete deck supporting the same live loads.

- **Rapid installation:** aluminum bridge deck panels can be installed faster than any other system; with a shop installed wearing surface, the light weight panels are quickly fabricated and erected; prefabrication, with no structural field welding, makes rapid installation and replacement possible.

- **High durability and low maintenance:** built from a corrosion-resistant material, aluminum deck does not require painting or protective coating.

5.2.2 Applications

Table 5.1 lists some of the aluminum bridges built in North America during previous decades (1930's through 1960's). It does not include the Smithfield Street bridge referred to earlier. Although wearing surface problems were observed, the aluminum alloys performed well. The Federal Highway Administration will be assessing the durability of the wearing surface in the Alumadeck system [5.3].

Table 5.1 Aluminum Bridges in North America [5.1].

Bridge Name	Year Built
ALCOA Railroad Bridge, Massena, NY	1946
Des Moines, Iowa Bridge	1958
Long Island Expressway Bridge, New York	1960
Three bridges on Long Island, NY	1960 -1963
Sykesville, Maryland Bridge	1963
Petersburg, Virginia Bridge	1963
Arvida Arch Bridge, Canada	1950

5.2.3 The Corbin Bridge

Built in the 1930's, the 320 ft (97.5 m) long Corbin Suspension Bridge (Fig. 5.2) over Junita River west of Philadelphia is listed in the National Register of Historic Places. With a 7-ton posted capacity, it could not be used by heavy emergency vehicles that needed a 24 km detour [5.3].

The new deck was installed on the Corbin Bridge in the fall of 1996. Alumadeck, the lightweight aluminum panels selected by Huntingdon County officials as a deck replacement, had a total weight of 36 ton. This increased the bridge weight limit from 7 to 20 tons allowing emergency vehicles to access the bridge. Twenty two extruded deck panels, each measuring nearly 17 m^2, were fabricated off-site by Reynolds Metals Co. They had shop-installed two-part epoxy aggregate wearing surface that increased speed of the deck installation.

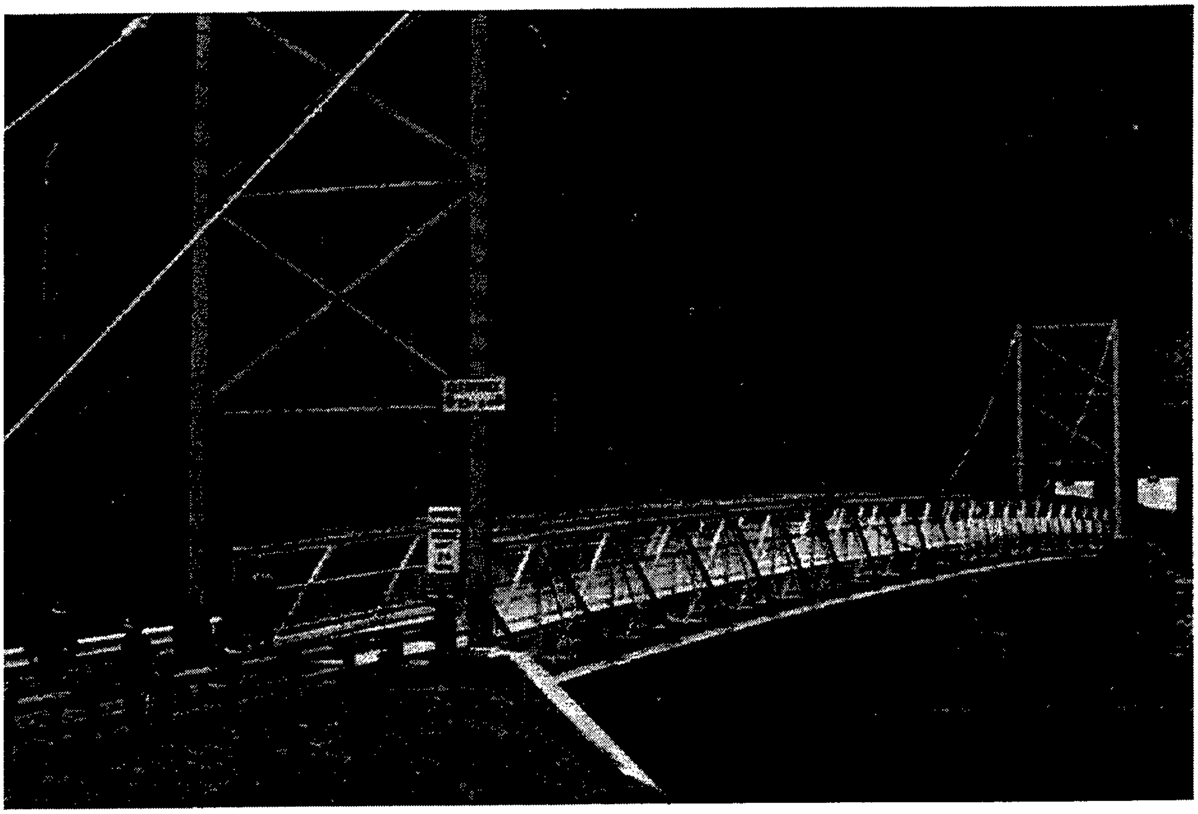

Figure 5.2 The Corbin Bridge [5.3].

5.2.4 The Little Buffalo Creek Bridge

The 67-year old, two-lane bridge, carrying Route 58 over Little Buffalo Creek in Clarksville, is the first in the state of Virginia to be renovated with an aluminum deck replacement system. The new deck, developed by Reynolds Metals Co., enabled the Route 58 bridge to be widened from 20 ft to 28 ft using the same foundations and weighing over 120,000 [lb] less than the old, narrower deck [5.1]

5.3 Overseas Aluminum Bridges

Over the past decade, about 40 bridges with aluminum decks have been built in Norway and Sweden [5.1]. More recently, the first aluminum road bridge was built over the Forsmo River, Norway some 35 miles (60 km) south of the Arctic Circle. The two-span, 39 m long, 7.4 m wide superstructure weighing 28 tonnes was fabricated in one piece in the workshop and transported to the site. The bridge has a twin box-girder section with inclined webs. It was opened to traffic in September 1995 [5.5].

"Speed of construction and installation (a few hours), low maintenance cost, low weight and the interest in testing new materials in bridge building were listed as reasons for using aluminum" [5.1]. Similar logic is the basis for the 300 m Smedasundet Bridge mentioned earlier. In this case, a cost analysis indicated that the aluminum bridge would be between 23-35% more expensive than a comparable bridge made of concrete or steel respectively.

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6. LIGHTWEIGHT CONCRETE

6.1 Introduction

The dead weight of long span bridges make up the largest part of the vertical loads the bridge must support. For example, in the 1991 m span Akashi-Kaikyo Bridge, dead loads were 91% of the total load [6.1]. Where spans fall between 200-600 ft (61-183 m), the proportion of dead load is lower but is still quite significant - between 70-85% of the total load in segmental box girders [6.2]. If the weight of the structure can be reduced, it can lead to overall savings.

Several alternatives means that may be used to reduce dead load are presented in Chapters 5-7. This chapter examines the use of lightweight concrete in bridge construction. The other chapters present solutions that can lead to reductions in the deck weight. Chapter 5 presented information on aluminum decks; Chapter 7 on the various types of proprietary lightweight decking that are available.

The United States has been using lightweight concrete in bridge construction since 1927. Over 400 lightweight concrete bridges have been constructed in twenty five states. Consequently, a great deal of information is already available regarding their long term performance. Florida has five lightweight concrete bridges, the first of which was constructed in 1963. There are, however, plans to build two others - a bascule bridge in Hallendale and another on the 17th Street Causeway near Miami [6.3].

This chapter provides a brief summary of information on lightweight concrete that is relevant for bridge construction. Much of the information is taken from a FHWA report [6.2]. Additional information may be found elsewhere, e.g. Refs 6.4-6.6.

6.2 Properties of Lightweight Concrete

Lightweight concrete typically uses processed aggregates such as expanded shales, clays, or slates that are obtained by heating the raw materials to high temperatures in a rotary kiln. The expanded aggregate is highly porous: the absorption of fully saturated expanded shale coarse aggregate may be 40-50% compared to less than 3% for normal weight concrete [6.2]. Consequently, mixing water must be appropriately adjusted during proportioning.

The aggregates are angular and have rough surfaces necessitating a higher percentage of fines. Mix design is established by trial mixes rather than by arbitrary specification of quantities. A brief summary of important properties of light weight concrete as reported in the FHWA report [6.2] are included for completeness.

Unit Weight

Structural lightweight concretes are concretes having a 28-day compressive strength of above 2500 psi (17MPa) and a air-dry density not exceeding 115-120 pcf (1840-1920 kg/m^3). This is between 76-80% of the density of normal weight concrete. Average air-dry unit weight of lightweight concrete ranges from 100 to 110 pcf (1600-1760 kg/m^3) and 105-120 pcf (1680 to 1920 kg/m^3) for concrete using normal-weight sand.

Strength

Compressive strengths of 5000 psi (35 MPa) can be easily achieved. Higher strengths of around 7000 psi (50 MPa) require high-quality aggregate while strengths of around 11,000 psi (77 MPa) require special admixtures such as silica fume and superplasticizers. High strengths are achievable by replacing lightweight fine aggregate by normal weight sand. Tensile strengths are lower: the splitting strength is approximately 70-100% that of normal weight concrete having the same compressive strength. This lower value also determines the shear stresses carried by lightweight concrete.

Bond Strength

Bond strength measured from pullout tests vary between 600-800 psi (4-5 MPa). Development lengths are greater because of the lower tensile strength.

Thermal Properties

The thermal expansion coefficient for lightweight concrete is lower than that for normal weight concrete and ranges between 4-6 x 10^{-6} per °F (7-11 x 10^{-6} per °C)

Creep and Shrinkage

Creep values are generally higher than normal weight concrete of comparable strength. Shrinkage values can be lower. Field measurements of creep and shrinkage tend to be lower than those obtained from laboratory tests on small specimens.

Skid Resistance

Skid resistance in lightweight concrete is higher because of the rough texture of the aggregate. Unlike limestone and gravel, lightweight aggregates do not lose their skid resistance over time because worn out surfaces are replaced by new rough surfaces exposed through wear and tear.

Elastic Modulus

The unit weight of concrete is factored into the AASHTO expression for elastic modulus. This, however, "consistently over estimates the values obtained from tests on high-strength lightweight concrete" [6.2]. It is recommended that certified test values provided by local aggregate producers be used during design.

6.3 Lightweight-Concrete Bridges

As part of the FHWA study [6.2], twelve lightweight concrete bridges from ten states were inspected during 1983-85 to assess their condition and also to gage the acceptance and experience of agencies that used them.

Table 6.1 provides a brief summary of the survey. In general, the performance of lightweight concrete was better than that of normal weight concrete exposed to the same environment. Louisiana and Texas both reported problems but these were attributed to the quality of the aggregate, construction procedures and the quantity of mild steel reinforcement used. More details may be found in Ref. 6.2.

Two of the earliest lightweight concrete bridges constructed in Florida - the Suwanee River Bridge at Fanning Springs (Table 6.1) and the Sebastian Inlet Bridge - Route A1A, Vero Beach constructed in 1964 were re-evaluated in an-depth investigtion conducted in 1992 [6.6]. Comparison of the test data with those recorded in 1968 indicated no deterioration with strain and deflection data being essentially same. Wearing characteristics of the exposed lightweight aggregate were found to be similar to that of normal concrete surfaces after 30 years of exposure.

Florida has only five lightweight concrete bridges. As was pointed out in the FHWA study [6.2] "Although the lightweight-concrete bridge structures built in Florida have been successful, apparently the construction control required for lightweight concrete and the competition of natural aggregate suppliers have discouraged its use."

Table 6.1 Survey of Selected Lightweight Bridges [6.2].

State	Bridge	Construction	Comments
California	Oakland Bay Bridge	1936	Deck in satisfactory condition after 48 years. Normal weight deck in approach signs were more chloride contaminated and had spalled.
	Parrots Ferry Bridge	1979	Largest and most complex bridge project using lightweight concrete.
	Napa River Bridge	1977	Satisfactory performance.
Florida	Suwannee River Bridge	1963	Excellent condition.
Maryland	William Preston Lane, Jr Memorial Bridge	1952 1973 (2nd bridge)	Deterioration in normal weight beam after 23 years. Chloride contamination in lightweight deck but no sign of deterioration.
New York	Coxsakie Bridge	1972	Excellent condition despite multiple deicing salt application.
Tennessee	I-24 over SR 76	1976	Excellent condition
Virginia	Woodrow Wilson	1962; light weight re-decking '83	Replacement deemed successful
Louisiana	US 190 over Amite River	-	Lightweight deck has proven satisfactory for 20 years
	US 61 over Thomson Creek	1961	Performing well after 20 years of service
Missouri	Jefferson City Bridge	1953	Deck performed well for 30 years
	Broadway Bridge	1957	Lightweight deck replaced after satisfactory 20 year performance by another lightweight deck
South Dakota	Sioux Falls, I-29	late 1950's	Lightweight performed better than normal weight concrete
Texas	Capitol Avenue over Buffalo Bayou	1962	Deck performed poorly- attributed to poor construction and quality control
	Haskell Overpass	1964	Performance quite good - sufficiency rating was 75.3 out of a possible 100

6.4 Durability of Lightweight Concrete

In normal weight concrete, the interface between aggregates and the cement matrix has a high porosity and a weak bond. This makes it susceptible to micro-cracking from tensile stresses arising from drying shrinkage or temperature changes. The weakness of the interface is responsible for poor load transfer. Not surprisingly, a 8,000 psi (55 MPa) cement matrix mixed with a 15,000 psi (105 MPa) coarse aggregate results in a concrete with a composite strength of 5000 psi (35 MPa) [6.7, 6.8].

Micro-cracking makes normal weight concrete more permeable and this adversely affects its durability. In contrast, the porosity of light weight aggregates enables it to serve as reservoirs of moisture prior to hydration that assists in "internal curing". The rough surface of the aggregate coupled with the availability of absorbed moisture allows better bond between the aggregate and the cement paste. Thus, the region between the aggregate and cement matrix is a true transition zone unlike in normal weight concrete where it is a zone of weakness.

The compatibility between of the matrix and the aggregates in lightweight concrete increases its tensile strain capacity, its resistance to cracking and its long term durability because of reduced permeability. This improved durability has been noted in inspections such as reported in Table 6.1.

6.5 Economics

The use of lightweight concrete can make structures between 20-40% lighter. Dead load reductions lead to lower foundation costs and reduced steel. As a result, despite the higher cost of lightweight concrete, overall savings may be realized.

In an attempt to quantify potential cost reductions, in the FHWA study five different bridge types (see Fig. 6.1) were studied. These included (1) single-cell box girders (2) steel plate girders incorporating composite decks (3) AASHTO I girders pretensioned with composite concrete decks (4) non-prismatic segmental cantilever box girders and (5) concrete cable stayed bridges. A two-span, three-lane continuous bridge with a deck width of 40 ft (12.2m) was assumed for the purposes of comparison. Several span lengths ranging between 50-1500 ft were investigated.

Table 6.2 summarizes the approximate savings in structural steel or prestressing steel obtained from the study. These varied between 13.5% for the cable stayed bridge to 21.4% for the segmental box-girder. These savings translate into more economical structures if they can offset the higher material cost of lightweight concrete.

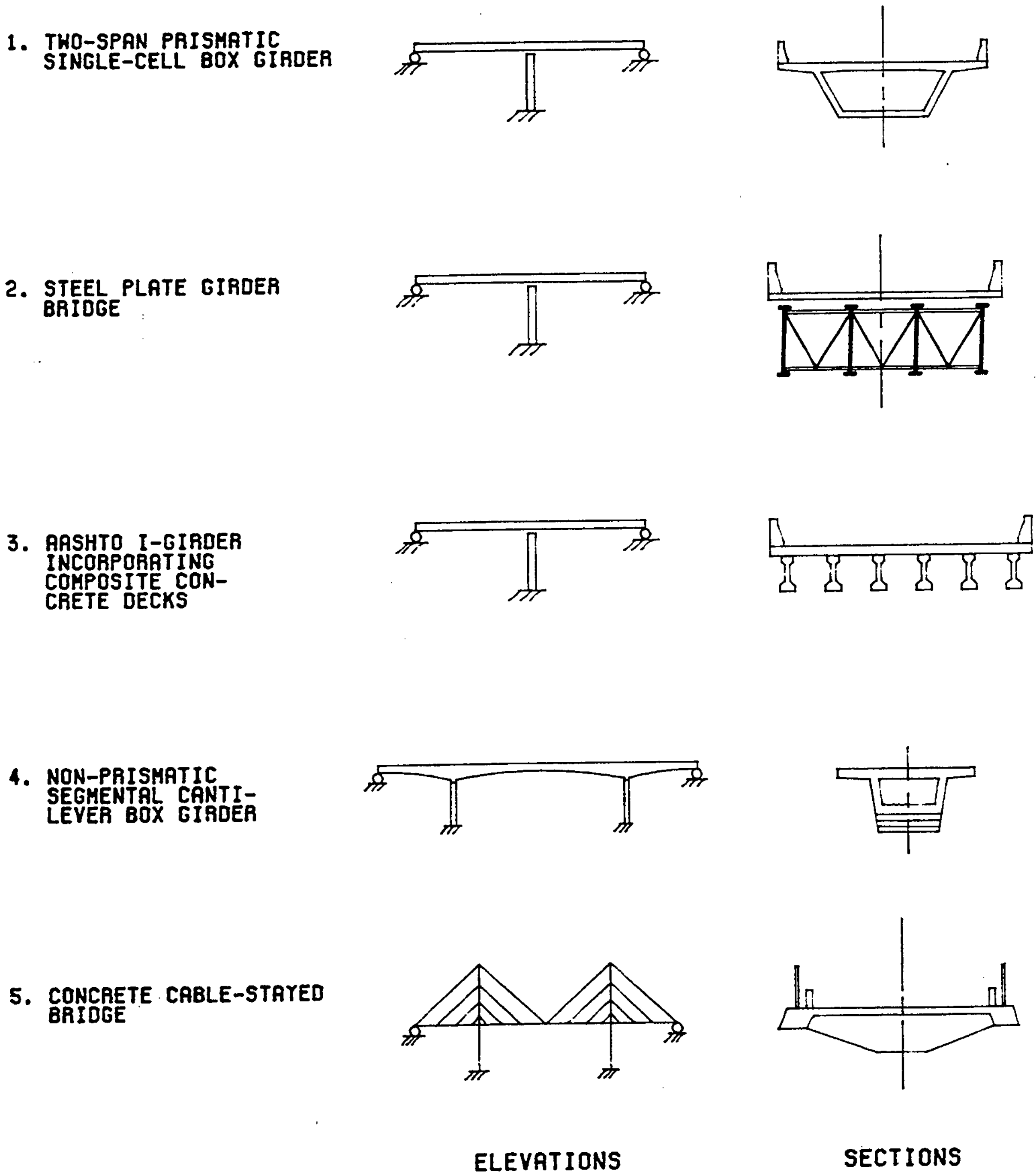

Figure 6.1 Systems investigated in FHWA study [6.2]

Table 6.2 Savings in Steel Due to Use of Lightweight Concrete [6.2].

Bridge Type	Structural Steel Reduction (%)	Prestressing Steel Reduction (%)
Single-cell box girder	NA	16.6
Steel plate girder	13.7	NA
AASHTO I-girder	NA	18.6
Segmental box girder	NA	21.4
Cable-stayed girder	Na	13.5

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7. LIGHTWEIGHT DECKING

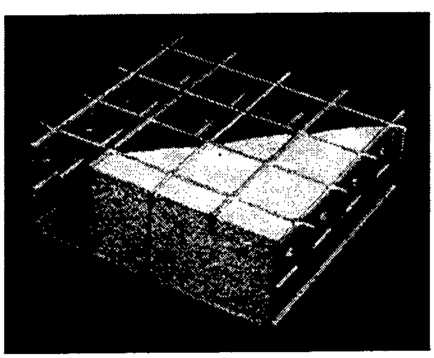
7.1 Introduction

This chapter presents an overview of alternative lightweight decking systems that may be used to reduce dead weight in long span structures. Several of the systems are proprietary and information presented was taken from the manufacturer's literature. The inclusion of this information in this chapter does not necessarily imply an endorsement of the system.

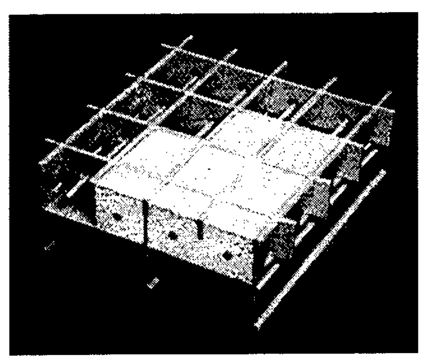
Section 7.2 describes steel grid decking system first used in the 1920's and Section 7.3 its evolution to the exodermic decking system in 1980. Two recently developed systems, the Harspan PT system and advanced composite decking are described in Sections 7.4 and 7.5 respectively. A comparison of the weight reductions possible from the various systems is summarized in Section 7.6.

7.2 Steel Grid Decking

Steel grids are an interlocking network of steel members supplied in large, modular customized panels that may be used in either new construction or in deck replacement. The grid may be used in combination with concrete in three ways (1) a full depth grid (2) a half filled grid and (3) an exodermic deck, described in the next section [7.1]. Several manufacturers offer variety of styles of the steel grid decking system. Figs 7.1-7.2 show examples of full-depth and half-filled grids from one of the manufacturers [7.2].



Full Depth



Half Filled

Figure 7.1 Full and Half Filled Steel Grid [7.2].

In the full depth grid, the concrete retaining pan is at the bottom. Common available sizes are 3 in., 4 1/4 in. and 5 3/16 in. The half-filled grid, is only available in the 5 3/16 size. In this case, the concrete depth is limited to approximately 2 ½ in and saves about 25 psf of dead load compared to the full-depth grid.

Open grid decks, i.e. grids not filled with concrete, are widely used in movable bridges. They do not provide a high quality riding surface, but offer time-efficient installation and weight savings. They weigh between 15 lb/ft^2 and 30 lb/ft^2. Half filled grid-reinforced concrete decks are heavier then open grid but still offer excellent alternative for constructing a lightweight, economical bridge deck weighing between 40 lb/ft^2 and 50 lb/ft^2. The grid-reinforced concrete decks filled full depth with the concrete are the heaviest but the strongest and most rigid. Their weight varies between 60 lb/ft^2 and 80 lb/ft^2 [7.2].

Used in conjunction with lightweight concrete, even greater weight reductions are possible. Examples of bridges where steel grids have been used with lightweight concrete are listed below [7.3].

- Mackinac Bridge constructed in 1956
- Rio Grande Bridge in New Mexico, 1966
- Two bridges over Cape Cod Canal: Sagamore Bridge, 1978 and Bourne Bridge, 1981
- Platt Bridge over Schuykill River in Philadelphia, 1985
- Tarentum Bridge in Pennsylvania, 1986
- Martin Luther King Bridge over Mississippi River, 1989
- Queensboro Bridge, 1994, New York City
- Pulaski Bridge, 1994, New York City

The practice of constructing bridge decks with steel grid application has been in use since the 1920's. With over sixty years of its utilization, the steel grid has undergone several enhancements [7.3]. These include the following:

- ***Design*** - originally designed as simple beams, they are now being analyzed as orthotropic plates. This will permit longer allowable spans. A compute program is available for this purpose.
- ***Attachment*** - the costly practice of welding grids to the framing element has been replaced. Currently, shear studs are used to secure the grid to the deck that also ensures composite action.
- ***Wearing surface*** - originally, the concrete was made flush with top of the steel grid. The wear of the concrete within the grid led to formation of "cups" that resulted in very poor

ride quality. Consequently, a suitable overlay is provided over the grid-reinforced concrete deck installed on highway bridges.

- ***Corrosion protection*** - Galvanized grid panels may be used to enhance corrosion resistance.

Open-Grid Advantages [7.3]

- Light weight.
- High durability.
- Strength: Independent laboratory tests have proven open grid's effective load distribution and excellent recovery capability.
- Quick installation: the prefabricated deck panels arrive at the construction site ready for installation by laying them in place and attaching to the bridge; installation can be completed in a matter of days.
- Simple construction: with an open grid deck there is no need for asphalt, concrete, drainage system or expansion joints.
- Minimum traffic disruption: open grid panels are capable of handling construction traffic right after their placement on the bridge; therefore, there is no need for disruption other lanes traffic.
- Good quality control: all of the deck manufacturing is performed in a factory; therefore, far more rigid adherence to specified tolerances can be achieved that it is possible to reach under job-site conditions.

RC Grid Advantages [7.3]

- Lightweight and high strength: weighing less than conventional reinforced concrete decks, a grid-reinforced concrete deck can have a major beneficial effect on the overall design of a new bridge. It can also serve as an economical solution for renovating a deteriorated bridge.
- Composite function: currently used attachment methods ensure composite action between the deck and supporting elements thereby contributing to the structural efficiency of the overall design.

- No formwork: For cast-in-place option, once a panel is correctly fastened in place on the bridge, it is ready to be filled with the concrete; since it has its own forms, no additional formwork is required and all of the labor and additional materials associated with cast-in-place conventional reinforced concrete deck are eliminated.

- Rapid deck construction: A grid-reinforced concrete deck can speed the installation process and minimize traffic interruptions; additionally, because the steel grid itself contains load bearing members, it forms a work platform for cranes and other work vehicles.

- High cost-to-performance ratio: the long life expectancy (up to fifty years, approximately twice as long as conventional reinforced concrete deck) makes grid-reinforced concrete decks economical.

- High durability: grid-reinforced concrete decks provide effective load distribution, minimal deflection and excellent recovery capability.

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7.3 Exodermic Deck

The exodermic (*or composite unfilled steel grid*) deck combines a steel grid, concrete and reinforcing bars to make the best use of each individual component. It consists of a reinforced concrete slab cast on top of a steel grid with the form pans positioned on top of the grid as shown in Fig. 7.2. It is made composite with the bridge's supporting superstructure through the embedment of headed studs with cast-in-place concrete, poured full depth over stringers and/or floor beams. No field welding of the grid panels to supporting beams, or grid panels to each other, are required.

Exodermic decks can be specified to accommodate a particular requirement of the bridge with overall deck thickness from 6 ¾ in. to 10 in. The concrete component can be precast (as in the Milton-Madison Bridge and Pitman Creek Bridge), or cast-in-place (as in the Route 9W bridge over Popolopen Creek and County Road 61 Bridge over Mohawk River / Barge Canal in St. Johnsville, N.Y.). The concrete can be textured to form an integral, monolithic wearing surface. If desired, membrane and/or overlay can be applied as a separate wearing surface.

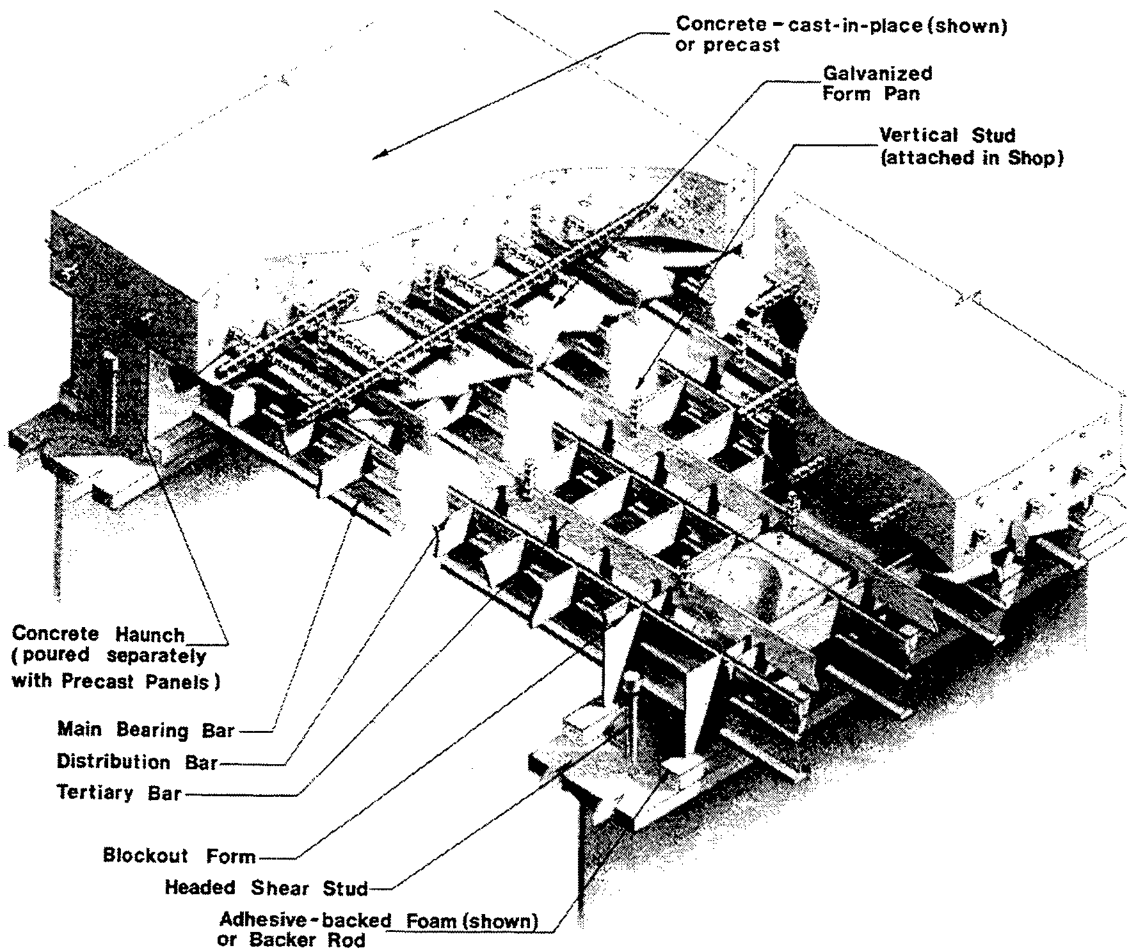


Figure 7.2. Exodermic Deck Elements [7.4].

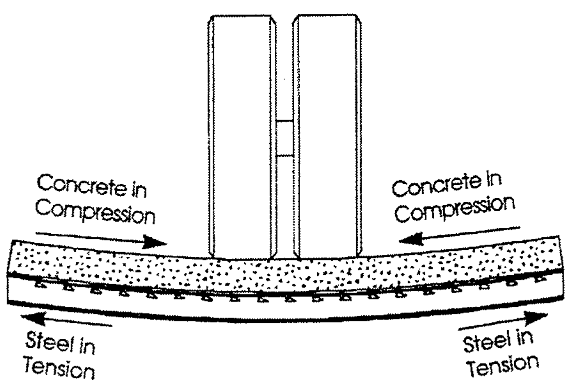
Deck Elements

- Main Bearing Bars - rolled shapes defining the strong direction of the grid, typically spaced from 6 in to 12 in; they are punched to permit interlocking with distribution bars.
- Distribution Bars - flat bars, usually spaced 4 in apart, punched to interlock with the main bars and tertiary bars.
- Tertiary Bars - welded to distribution bars; they extend into the concrete (usually 1 in), and provide composite action between the steel grid and reinforced concrete.

- Vertical Studs - short vertical pieces of reinforcing bar acting to counter vertical separation forces; typically 2 ¼ in long, they are welded to tertiary bars at 12 [in] on center.
- Haunch - area over supporting beam; filled in full depth with concrete embedding welded or bolted shear studs to assure composite action between the deck and its supports. The haunch concrete is always placed in field.
- Form Pan - usually 20 gauge galvanized sheet steel preventing concrete from filling the steel grid.
- Reinforcing - usually epoxy coated for corrosion protection, permits an exodermic deck to resist negative moments in cantilever deck or over the beams in continuous deck.
- Concrete - usually 3 in to 5 in thick, with recommended low water-to-cement ratio and 3/8 in max. coarse aggregate; if deck weight is critical, lightweight concrete can be specified.
- Headed Shear Stud - embedded in haunch concrete, they assuring full composite behavior between deck and superstructure; shear connectors should extend minimum 2 in into the steel grid.

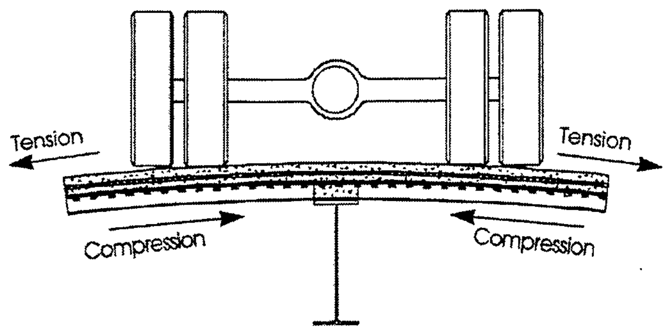
Structural Behavior

By design, an exodermic deck makes the best use of its two components, reinforced concrete and steel grid using them more efficiently than in the standard reinforced concrete slab. Thus, an exodermic deck can be substantially lighter without sacrificing deck's stiffness or strength.



All concrete is in compression and contributes fully to the section. The main bearing bars of the grid handle all tensile forces at the bottom of the deck.

Figure 7.3 Exodermic Deck in Positive Bending [Ref. 7.4].



Reinforcing bars in the top portion of the deck carry the tensile forces; compressive forces are handled by the grid main bearing bars and the full depth of concrete placed over all stringers and floorbeams.

Figure 7.4. Exodermic Deck in Negative Bending [7.4].

Advantages

- Rapid bridge construction with possibility of maintaining all traffic lanes during the peak hours.
- Light weight (exodermic deck typically weights 50% to 65% of a standard reinforced concrete deck of the same overall thickness).
- Higher stiffness and strength than a standard reinforced concrete deck of equal weight.
- Long service life - 50 years life is a reasonable life expectation for an exodermic deck.
- Good riding surface without additional overlays.

Precast Application [7.5, 7.6]

Precast exodermic deck are used where traffic conditions require rapid deck replacement, very often conducted during a nighttime work window. Such a schedule of work permits the structure to be fully open to traffic during daytime. Examples of precast exodermic decks include the Driscoll Bridge on the Garden State Parkway, the Pitman Creek Bridge and Millton-Madison Bridge in Kentucky, emergency repairs on the Tappan Zee Bridge over the Hudson River and the Troy-Menands Bridge in Albany, NY.

Cast-inPlace Application [7.4]

Exodermic deck with its concrete part cast-in-place can be installed if traffic permits longer bridge closing time. This construction method was used for the first time in 1991 on the Popolopen Creek Bridge in Highlands Falls, NY. Using steel grid as a rapidly-placeable, stay-in-place form, substantial savings were realized over conventional concrete deck framing. Other examples include the Mohawk River Bridge in St. Johnsville, NY, the Calumet Street Bridge in Philadelphia, and the Clover Run Bridge in St. George, WV.

New projects include Gateway Bridge (17th St. Causeway) in Ft. Lauderdale, Florida, where E.C. Driver have selected an exodermic deck for the new, twin, double leaf bascule project. The cast-in-place exodermic deck will span between 14.4 ft o.c. floorbeams.

References

7.4. Exodermic Bridge Deck (1996)._ Exodermic Bridge Deck Institute, Inc.

7.5. Bettigole, Robert A. (1994). "Exodermic Deck Allows Nighttime Redecking of Kentucky Bridge", Exodermic Bridge Deck Institute, Inc., Summer.

7.6. ENR (1997). Exodermic Deck System Lightens Load for Kentucky, February 24.

7.4 Harspan PT

Harspan PT is a newly developed deck system suitable for new construction or deck replacement that can be precast or cast-in-situ. In this deck, a 3.5 in. low slump, high density concrete slab is composite with 6 in. deep transverse bars (*small beams of dumbell cross-section*) spaced at 16 in centers.

The slab is haunched to a full structural depth of 8 in. at the supporting beams or girders to accommodate ducts for longitudinal post-tensioning of the deck (see Fig. 7.5). The post-tensioning eliminates the need for reinforcement and also enhances composite action between the bars and the slab without shear connectors. The deck is also composite with the main beams or girders made of steel, concrete or wood.

The deck is made of 8 ft long panels that can cover the full width of the deck upto 60 ft. The length of the end panel is variable to match the length of the bridge. The steel bars are individually cold bent to agree with the cross-fall of the deck. On an average, the deck weighs 55 lb/ft^2. The deck was designed to compete both performance and cost-wise with regular 8 in. concrete decks.

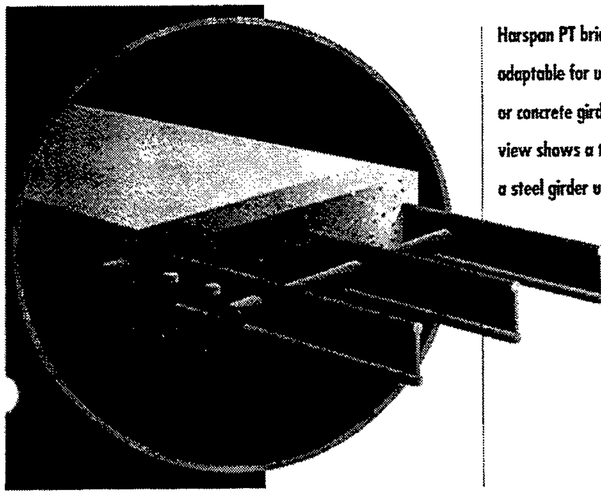


Figure 7.5 Harspan PT Bridge Deck [7.8].

Harspan PT was extensively tested at the University of Pittsburgh. Based on the test results, it was concluded that post-tensioning eliminated the need for any reinforcement in the concrete, prevented the formation of transverse surface cracking and decreased the tendency for longitudinal surface cracking [7.7-7.8].

References

7.7 Casagoly, P. (1997). "Testing of a Composite Steel-Concrete Bridge Deck", Concrete International, Vol. February 1997.

7.8 IKG Borden (1995). Brochure on Harspan PT.

7.5 Advanced Composite Deck

A new lightweight fiber reinforced polymer (FRP) deck has been developed that has six to seven times the load capacity of a reinforced deck while being only 20% its weight. This modular deck with a double-trapezoid or hexagon cross-section may be used for new construction or for speedy replacement of deteriorated decks.

The FRP deck was designed and tested under the Construction Productivity Advancement Research (CPAR) program of the US Army Corps of Engineers with the main participants being West Virginia University's Constructed Facilities Center, the Construction Engineering Research Laboratory in Champaign, Illinois, the Composite Institute of the Society of Plastics Industry in New York City and Creative Pultrusions of Alum Bank Pennsylvania. [7.9]

Examples of demonstration bridges constructed are provided in Chapter 10 and are therefore not repeated here. Although FRPs have significant advantages over conventional materials, the major obstacles to their use are their high initial cost and *conceived* risk of failure. Life cycle cost analysis carried out indicates that FRP decks can compete with a conventional concrete deck if it is durable. According to this analysis, the owner cost for a FRP deck is only 1% more than that for a conventional deck for a 75 year life expectancy. Over a 50 year life expectancy, FRP costs 11% more [7.10].

References

7.9 Staff (1997). "Superdeck's Debut", Bridge Design and Engineering, No. 9, November, pp. 79-80.

7.10 Lopez-Anido, R. (1998). "Technical and Cost Analysis Evaluation Modular FRP Composite Bridge Deck", Draft Report dated March 4, West Virginia University, WV.

7.6 Comparisons

A summary of the weight comparisons of the various decks is presented in Table 7.1.

Table 7.1 Weight Reductions for Various Deck Systems.

System	Weight (lb/ft^2)	Comments
Normal Weight Concrete	100	8 in. slab
Lightweight Concrete	75	Assuming 115 pcf density
Aluminum	20	Maximum beam spacing - 12 ft
Steel Grid Decking		
Open	15-30	
Half-Filled	40-50	Greater reductions possible with lightweight concrete
Full Depth	60-80	
Exodermic Decking	50-65	
Harspan PT	55	
FRP Deck	20	Maximum beam spacing - 9 ft

8. THE USE OF SMART TECHNOLOGY IN BRIDGE ENGINEERING[1]

8.1 Introduction

The study of smart materials and structures is a field that has been cited by Scientific American as one of the key technologies for the 21st century [8.1]. A structure or material can be considered "smart" when it has the ability to sense internal or external conditions and respond in some manner appropriate to alter the effects of those conditions in a favorable way.

Smart technology has been around for several decades, debuting in the early 1960's when Corning Glass Works started developing a new kind of glass [8.2]. This new "photochromic" glass was able to react to the amount of light present in its environment - automatically darkening in the light and automatically lightening in the dark. This is the same glass that now makes up the large number of sunglasses with variable and self-adjusting transparency. The application of photochromic glass was not limited to sunglasses. It was soon expanded to include glass in buildings and automobiles and was considered for just about every application where glass or mirror was needed [8.2]. Soon after the introduction of photochromic glass, scientist, researchers and engineers began to recognize the potential for such smart technology. A material that could respond to its environment could be useful far beyond the glass industry. Designs incorporating piezo-electric ceramics and shape memory alloys were a few of the innovations to comprise the next wave of these new smart materials and structures [8.3]. More and more, designs are sought for materials and structures that not only serve the purpose for which they are contrived, but do so actively in the most efficient manner possible. Smart technology is not only a new kind of technology, it is a new way of thinking.

Smart technology quickly found its way into the designs of the aeronautical, aerospace and mechanical engineers [8.4]. However, civil engineers are notorious for abiding by methods that are tried-and-tested, and for this reason smart technology is only slowly making its way into the civil engineering community. The most notable examples of smart designs in civil engineering are the active or intelligent buildings, several of which exist in Japan and only one or two of which exist in the United States [8.5]. These buildings detect movements or vibrations caused by winds or earthquakes, and respond accordingly using damping systems that reduce the effects of such stimuli. Similar designs have been proposed to reduce the wind-induced vibrations on long-span bridges. However, unlike the active building systems, this smart design for bridges has not yet found its way into practical use.

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[1] Excerpted from paper by Mr. Mike Slaven that was the recipient of the best undergraduate paper award at the 15th International Bridge Conference, Pittsburgh, June 1998

While smart technology has not yet made its way into the practical designs of the bridge engineer, it has been a major topic of the research community. Test bridges have been interlaced with fiber optics that can detect stresses or strains, and signal if failure is imminent [8.6,8.7]. Others have been equipped with piezoelectric ceramics that can monitor vibrations and signal actuators to change the mechanical properties of the bridge, thus reducing or eliminating the damaging vibrations [8.8]. Some researchers have focused on the use of sensors that can detect corrosion in rebar, and signal for repair if possible, and replacement if not [8.9]. Others have designed concrete that can mend itself when minute cracks are formed [8.10]. These are just a few of the examples of research that is currently being undertaken. Many others currently exist, and even more are likely to exist in the future as the benefits of smart technology become more widely known and accepted.

Smart technology, like any newly emerging technology, is not without sizable cost. It is obvious that a smart bridge with fiber optics, sensors, actuators and other "smart parts" would be more expensive than its "dumb" counterpart. But when the big picture is considered, the economical feasibility of a smart bridge becomes evident; not only are resources such as concrete and steel reduced in a smart bridge, but more importantly the cost of service and maintenance is reduced while the service life is greatly increased. Furthermore, the technology itself will decrease in cost as it becomes more widely utilized, thus making smart design more economically feasible, and hopefully even more so than current design standards.

In the sections that follow, just what is meant when a structure or material is classified as "smart" will be defined followed by some examples of existing smart materials and structures. Current and developing applications of smart technology in bridge design will then be described, along with accounts of related research activities. Then, other possibilities for the future of bridge design will be suggested. Finally, the use of smart technology in bridge design will be presented as not only beneficial, but as indispensable as we cross over into the 21st century.

8.2 What Makes a Material or Structure Smart?

A material or structure can be considered "smart" when it has the ability to sense internal or external conditions and respond in some manner appropriate to alter the effects of those conditions in a favorable way. The difference between smart materials and smart structures is vague. Many smart structures are smart only so far as the materials they are comprised of are smart. Likewise, the "intelligence" of a lone smart material that is not incorporated in an overall structural system is contestable. Therefore the term "smart technology" has come to encompass not only smart materials and smart structures, but also the way of thinking in which the desired result is a system, structural or otherwise, that can detect and respond to external and/or internal stimuli. This can be as simple as the toilet that senses you are done with your business and automatically flushes, or it can be as complex as the multi-story building that can detect large vibrations and counter them by actively altering the stiffness of individual structural members.

In general, smart systems include three basic components or ideas: (1) Sensors, or the ability to sense or detect important internal or external information; (2) actuators, or the ability to respond, react, or in some way alter the state of the system; and (3) a control mechanism to act as the brain of the system, interpreting the information gathered by the sensors and notifying the actuators of the best possible actions to take. [8.11] Note that in some systems, the only role of the actuator is to inform some third party of the information the sensors have collected, and not alter the system itself.

For this system to be truly smart, it must "think" all the time. There can be no such thing as a system that is only smart temporarily, or only smart intermittently. The toilet that automatically flushes itself must do so according to some pattern of logic, and it must follow this patterned logic at all times. A toilet that simply flushes every five minutes may serve the same purpose as a toilet that flushes automatically, but it can not be considered a smart system because it does not serve that purpose in the most efficient manner possible. A general pattern of logic that all smarts systems follow is illustrated below in Fig. 8.1. All processes start with the acquisition of information by the sensors. This information is then sent to some control mechanism for processing.

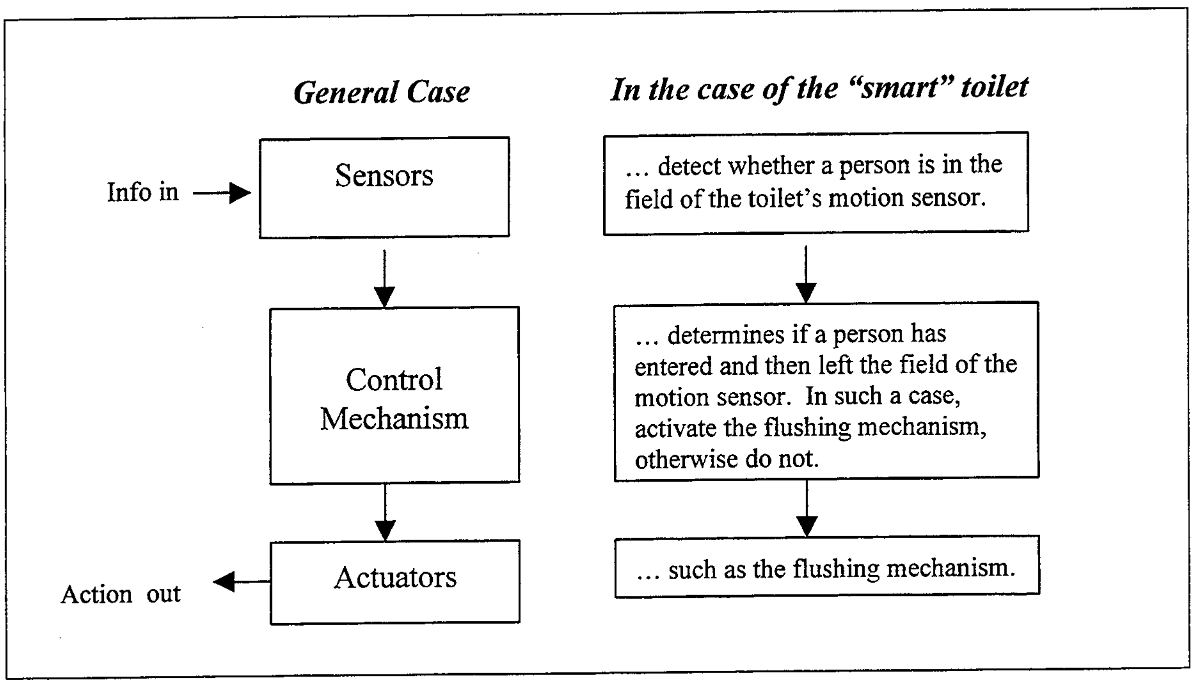

Figure 8.1 General Pattern of Logic in a Smart System.

The control mechanism is programmed to act on this information in a pre-specified way. If called-for, the control mechanism will signal the actuators to alter the system in some way. This entire process of sensing, control processing, and active response is repeated continuously in a smart system.

In order for this model to accurately represent all smart systems, each of the terms used should only be taken in its most generic form. By "sensor" it is meant anything that has the ability to collect any type of information, including material properties, mechanical properties, and environmental conditions, to name a few. By "actuator" it is meant anything that has the ability to produce an action, or otherwise alter the state, properties, or environment of a system. The action that the actuator commits does not have to be internal – it could be as simple as notifying someone or something that action is necessary. In any case, the end result is action. The control mechanism is anything that links the sensors to the actuators in a logically structured way. In some smart materials, each of these three necessary components may not be individually distinguishable, but instead inherently intertwined. Such is the case of the photochromic glass mentioned in the introduction. This glass has the ability to detect information about its environment, and react according to this information in a logically structured way – if the environment lightens the glass darkens, and if the environment darkens the glass lightens. But one may argue that this glass in no way has sensors, actuators, and a processing mechanism. In the following paragraphs, it will be illustrated how these components are indeed present in photochromic glass, as well as in several other smart materials and structures.

8.2.1 Existing Smart Materials

Generally accepted as the first smart material, photochromic glass, like many smart materials, operates at the atomic level. This self-adjusting glass was created by adding silver and copper halide to the traditional glass formula, creating tiny salty inclusions throughout the glass that are so small and transparent that they barely absorb or scatter any light. When light strikes these inclusions, an electron is ejected from the copper ions and adopted by the silver ions to form neutral silver atoms. These neutralized silver atoms create millions of tiny specs throughout the glass and thus create the light-blocking tint. When the light is removed, the process reverses and the glass clears again. [8.2] In this process, the three components that characterize a smart system are also at the atomic level. The copper ions can be considered sensors, by detecting the intensity of light present. The actuators are the silver ions, by accepting an electron and thereby changing their own properties, as well as the reflective properties of the glass they constitute. Most questionable is the existence of a control mechanism, but since there is a direct relation between the intensity of light and the reflectivity of the glass, a control mechanism is apparent. In this case, the nature of the copper and silver ions, and more generally the nature of atomic chemistry would be considered the control mechanism in this smart system.

There are many kinds of smart materials out there, but some show more potential, and more promise than others. *Research News* focuses on three main classes of smart materials that are bright on the horizon; (1) Piezoelectric ceramics and polymers, (2) shape memory alloys, and (3) electrorheological or magnetorheological fluids. [8.12] Piezoelectric ceramics can act as either pressure sensors or mechanical actuators. The electric polarity of their crystal structures allow them to quickly transform any mechanical forces into electric current, or conversely, transform electrical current into mechanical vibrations. They can produce these mechanical vibrations at very high frequencies, and thus are of utmost importance in the development of smart systems that counteract damaging vibrations.

Shape memory alloys are better suited for slower, stronger responses. Below a certain temperature, a shape memory alloy will take on any shape it is bent into. But when heated back above this temperature, it will try to return to its original shape. If something is hindering the restoration of the alloy's shape, it will exert a constant force. This force is the result of the atoms in the alloy attempting to toggle between different geometric arrangements [8.12].

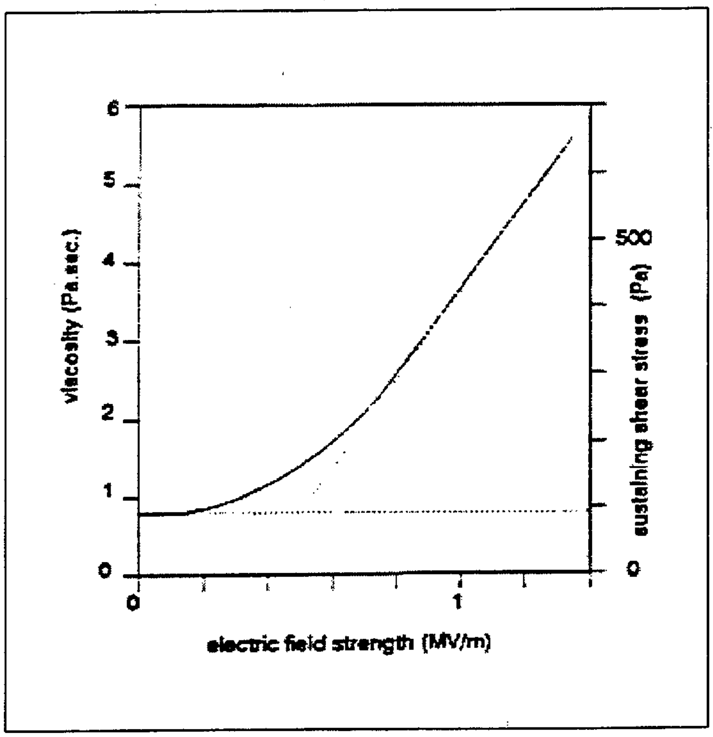

Figure 8.2 Properties of a Typical Electrorheological Fluid. [8.14]

An electrorheological fluid is a fluid whose viscous properties may be modified by applying an electric field, and a magnetorheological fluid is one whose viscous properties are modified by applying a magnetic field. This change in the viscosity of these actuator materials may be so extensive that they in effect change from liquid to solid. These fluids consist of fine polarizable particles of ceramic or polymer suspended in a liquid such as silicone oil. When an electric or magnetic field is applied, the particles "organize themselves into filaments and networks" [8.12], thereby stiffening the material. When the electric field is removed, the process reverses and the organization of the particles disappears – the material becomes fluid again. But as Culshaw points out, the resulting solid is useful only for transferring shear stresses. [8.13] Normal stresses allowed by the electrorheological or magnetorheo-logical solid are the same as those allowed by the liquid. Culshaw also explains how the viscosity and resulting shear strength of such a material increases almost linearly with the strength of the applied field, or is at least so when the field is relatively strong. Fig. 8.2 illustrates the properties of a typical electrorheological fluid as presented in Culshaw's book Smart Structures and Materials.

Of course the materials listed and explained here are just some examples of the possibilities allowed by smart material technology. But the potential of these smart materials is limited if one tries to utilize them outside the context of an overall smart system. Their true potential is realized only when they are incorporated in a system that can harness their intelligent abilities.

8.2.2 Existing Smart Structures

The difference between smart materials and smart structures can be seen as one of size. The three components that lend smart materials their intelligence operate on a microscopic scale, while those of the smart structures operate on a macroscopic scale. Often in a smart structure, the sensors, actuators and control mechanisms are items that are used regularly in other fields, but not in such a way that they create a smart system. A typical smart building designed to detect and counteract earthquake movements may include such items as accelerometers, actuators that operate on basic hydraulic principles, and a basic microcomputer as the control mechanism, to interpret the data gathered by the accelerometers and send the appropriate message to the hydraulic actuators. This type of smart structure is commonly known as an active bracing system.

A smart building has been erected on the campus of the University of Vermont incorporating fiber optics, thermistors, and strain gauges to collect different types of information from different parts of the structure. The sensors monitor the building for "cracks and vibrations due to wind, temperature changes, thermal expansion, and occupant loads." [8.15] This building does not employ any actuators that can bring about a change independent of human interaction, but the control mechanism – the microcomputer that logs the information gathered by the numerous sensors can double as an actuator by alarming when action needs to be taken.

It is obvious that the same type of smart technology described above is useful far beyond the monitoring of buildings. This same system and the same techniques can be used to monitor just about any structure made of materials, which includes every structure possible. The more researchers, scientists and engineers experiment with the ideas of smart technology, the more possibilities they uncover. They spread their findings to try and include other disciplines so that the overall benefit and value of their work is maximized. The bridge engineer has benefited from the idea of photochromic glass that was actualized at Corning so many years ago. Currently, one of the hottest topics among the research community in bridge engineering is the use and integration of smart technology.

8.3 Current Research for Smart Technology in Bridge Engineering

The application of smart materials and systems in bridge engineering has been researched more in the past few years than ever before. While the civil engineer is traditionally one who sticks to the 'tried-and-tested' philosophy, he/she is beginning to recognize the potential and feasibility of a smart bridge. Ideas for bridges that can monitor their own health, counteract corrosion at the moment it starts, and notify occupants of impending failure made their way from the minds of their creators, to the laboratories, to test beds in little relative time. In the following sections, some of these ideas, the most promising and highly researched areas of smart bridge technology, are presented.

8.3.1 Nerves of Glass

Fiber optic sensors are being used as the "glass nerves" [8.8] in today's experimental smart bridges. Fiber optics have the ability to detect a number of conditions, such as changes in temperature, pressure, stress and loading, or just about any other change in a physical or chemical condition. While there previously existed other sensors that could perform each of these individually, fiber optic sensors have some definite advantages. They are more accurate than the previous sensing methods available; they are smaller in weight an size; they are immune to electromagnetic interference and lightning; and they can be easily placed in a concrete structure before it is poured [8.10]. Fiber optic sensors can detect cracks forming in concrete before they are even visible to the naked eye. Even larger discontinuities may be present in a concrete mass that, due to their location are not detectable by ordinary methods. In a smart bridge of the fullest sense, the fiber optic sensors would serve as the sensor component in the three component smart system, sending the information it has gathered to the control mechanism, which would in turn send some command to an actuating device, or not depending on the input from the control mechanism. In any case, the signals from the optical fibers have the potential of providing an intimate and complete picture of a bridge's internal health.

Furthermore, bridges instrumented with fiber optic sensors could be monitored remotely from a central station or through telephone lines [8.16]. The costs associated with sending groups of inspectors out to monitor the health of these structures several times per year would be eliminated. One group of researchers has taken this idea one step further, and proposed that the information gathered by these fiber optic nerves be automatically encoded into HTML (hyper-text mark-up language) and made available for access over the internet [8.17]. This would in effect enable an engineer to monitor the status of a structure from anywhere in the world. Since microcomputers are usually used to interpret and log the data from the fiber optic sensors, there would be little additional cost in the added convenience of broadcasting this information over the internet. A diagram illustrating the interaction between the instrumented structure, the microcomputers and the internet is given in Fig. 8.3 on the following page.

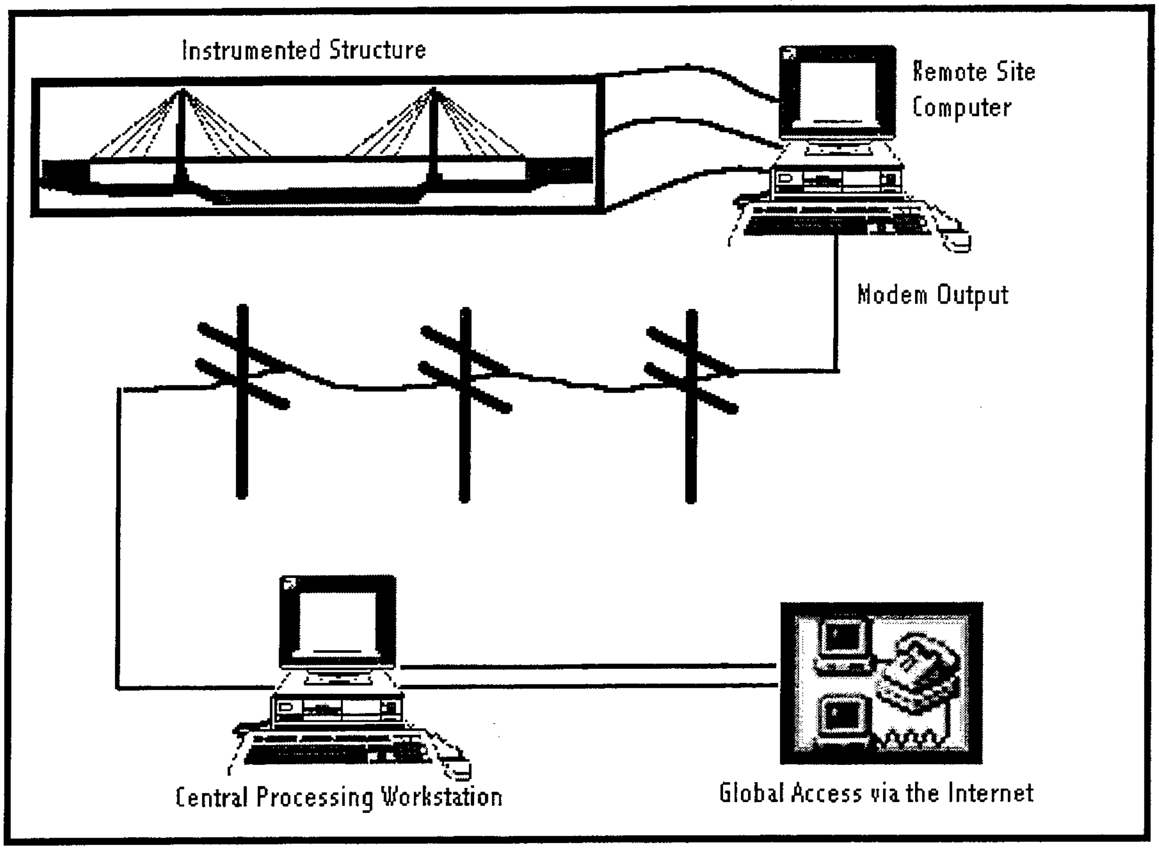

Figure 8.3 Computer Connectivity and Interaction Diagram for Monitoring of a Smart Structure via the Internet.

In such a configuration, the processing workstation calls the PC-based fiber optic sensing system at the remote structure, downloads the current data, processes and analyzes the data and produces summary reports compatible with HTML based internet browsing software [8.17].

8.3.2 Self Repairing Concrete

While the fiber optical nervous system mentioned above may be invaluable in detecting cracks, corrosion and other precursors to failure, it can do little on its own to alleviate such damage, outside of signaling for its repair. A materials scientist at the University of Illinois has developed and tested a simple idea that results in a type of "smart concrete" that can sense and repair its own damage [8.18]. To combat the cracking that leads to corrosion of reinforcement, small hollow brittle fibers are filled with a type of adhesive that has a concrete bonding ability. These fibers are placed throughout the concrete at the time it is poured, with special emphasis given to those locations that are especially prone to cracking. When a small amount of cracking does occur in the concrete, these small fibers also crack, releasing their adhesive and sealing the short-lived anomalies. The same methodology has proven useful to combat corrosion. The brittle fibers are replaced by fibers that corrode with relative ease compared to the concrete they reside in. At the first sign of corrosion, these fibers dissolve and instead of releasing adhesive, release an anticorrosion chemical that prolongs the life of the steel reinforcement. Tests were performed on this smart concrete by the same researchers who gave it birth, and the results revealed a higher compressive strength on a *second* severe loading resulting from the setting of adhesive released from the first of such test loads [8.18]. Figure 8.4 below summarizes how smart concrete senses and counteracts cracking and corrosion.

ENVIRONMENTAL DISTRESS TREATED	SENSORS		ACTUATORS	
	FIBER	MECHANISM TO RELEASE CHEMICAL	STIMULUS	RELEASE OF CHEMICAL
CRACKING	HOLLOW BRITTLE OR HOLLOW POROUS FLEXIBLE PERHAPS COATED (FIBER-GLASS POLY-PROPYLENE COATING OF WAX, CEMENT)	FIBER BREAKAGE COATING DEBONDING CONTRACTION OF FIBER	LOADING OR CRACK ENERGY	CROSSLINKER REHYDRATER POLYMER ADHESIVE (EPOXY, XYPEX, CEMENTS)
CORROSION	COATED, HOLLOW POROUS (POLY-PROPYLENE COATING OF POLYOL)	COATING DISSOLUTION	CHANGE IN PH OR PRESENCE OD CHLORIDE IONS	ANTI-CORROSION CHEMICAL (CALCIUM NITRATE)

Figure 8.4 Stresses in Concrete and Chemical Remedies [8.18].

8.3.3 Shape Memory Alloys

In traditional engineering design the most common way to increase the strength or rigidity of a structure was to add more material – give it a larger cross-section and load it up with more steel reinforcement. But by incorporating strands of shape-memory alloy, a structure's rigidity or stiffness can be altered by applying to the alloy heat or an electric current. Since a shape memory alloy by nature deforms when heat or electricity is applied, confining its shape produces a force instead of a displacement. This force results in a stiffness change that would be desired in such cases when variable stiffness can prevent collapse. Such a case exists when earthquakes or winds send a structure vibrating at its natural frequencies. Fiber optics or some other type of sensing network could recognize when such vibrations are beginning, and the smart system would signal the shape memory alloy strands to "flex", stiffening the structure and thereby canceling or minimizing the potentially damaging vibrations. In the case of long span bridges, this same methodology could be applied to counter the smaller traffic-induced vibrations that cause many fatigue problems. Researchers Shoureshi and Bell have even suggested that the shape memory alloys be integrated into the cables of large cable-stayed bridges [8.20]. In such a design, the incorporation of "active cables" would allow distributed tension control over the entire bridge.

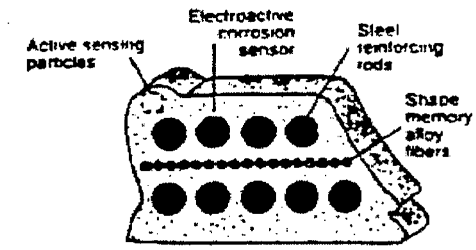

Figure 8.5 Smart Concrete with SMA's [8.19].

8.3.4 Strain Memory Alloys

In order for traditional strain gages to provide a "strain history" of a member such as a steel beam, they must (1) be fed an electric current at all times and (2) be connected to a device that cannot only interpret strain, but also monitor this strain so that the maximum is always known. Strain memory alloys, although they will not give a complete strain history, do have the inherent ability to "remember" the maximum strain they have been subjected to. When these alloys are subjected to a strain, they become irreversibly magnetic. The more they are strained, the more magnetic they become, and because the process is irreversible, the extent of magnetism relates only to the maximum amount of strain encountered. A company known as Strain Monitor Systems (SMS) has already developed this technology and is actively marketing it to the civil engineer. Fig. 8.6 on the following page is an excerpt from their site on the internet [8.24] that illustrates how the strain reading from a strain memory alloy will always reflect the peak strain encountered over the life of the alloy.

The innovations described so far, even though they are likely the key players in the future of smart bridges, are just a small sampling from the large number of possibilities currently being researched. At last check, there were four full-length journals being published at least annually devoted to only the topics of smart materials, smart structures and smart systems. It seems that every idea written about yesterday sparks two new ideas today, as the field continues to gain size and acceptance as a plausible design alternative.

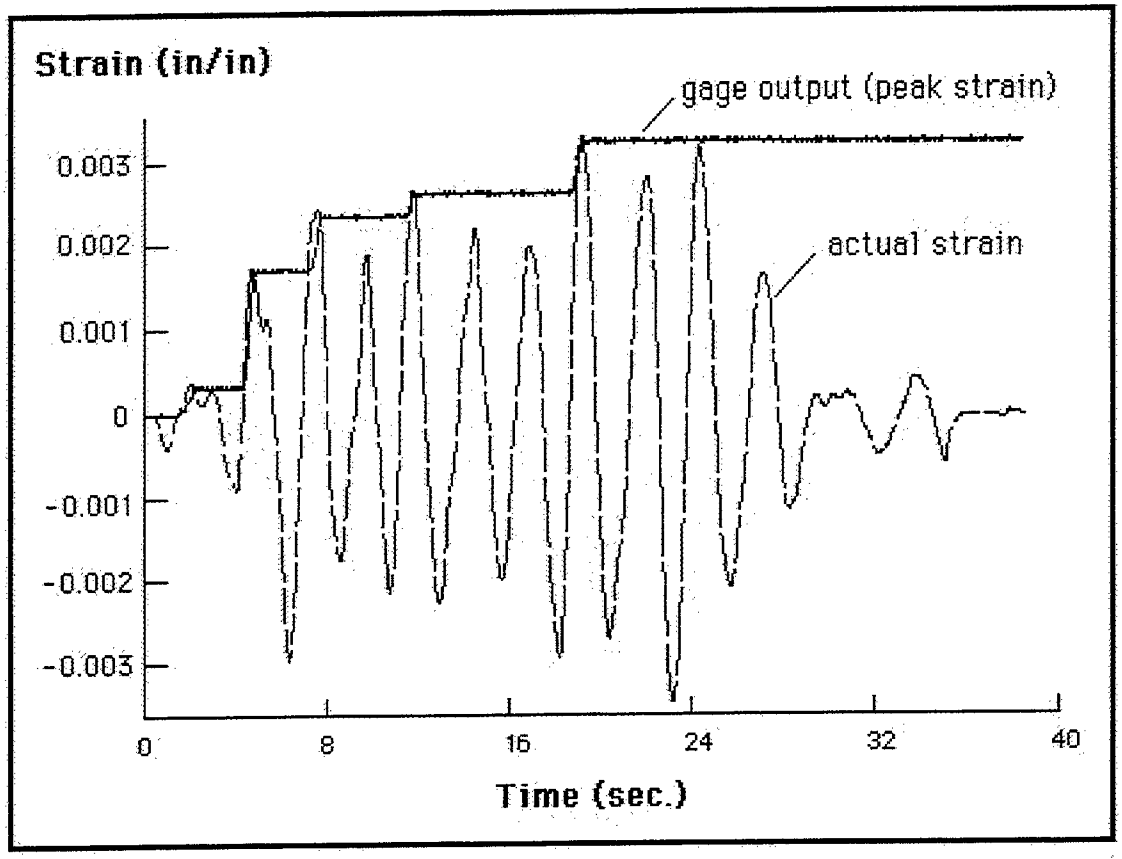

Figure 8.6 Actual and peak strain versus time for an earthquake induced displacement [8.24].

8.4 Existing Smart Bridges

Smart technology has a long way to go before reaching its fullest potential. Like any new technology, it must be tested over and over and demonstrate its own value. In bridge engineering, like any facet of civil engineering, there is a further requirement - this proof of value must be demonstrated not only to the scientist and the engineer, but also to the public. It is the public whose money generally pays for such projects and whose welfare depend on such projects. It is because of this that new technology in bridge design is slow in its ascent. Smart technology has just recently made its way out of the lab of the civil engineer and into the general public. Following are several case studies that illustrate bridge designs incorporating smart materials or systems that have been placed in real world situations – the first step toward widespread acceptance.

8.4.1 Vermont

Researchers at the University of Vermont, Burlington hope to lend new life to an old steel truss bridge by including an array of fiber optics during its reconstruction. The researchers call it "the most ambitious network of embedded fiber-optic sensors yet"[8.6] The sensors being incorporated into the reconstruction include not only strain sensors, but also vibration and corrosion sensors, so that the bridge's health can be fully diagnosed at any time.

8.4.2 Canada

The Headingley Bridge is a concrete bridge touted by the researchers at the Canadian Network of Centers of Excellence in Intelligent Sensing for Innovative Structures (ISIS) as the world's longest span bridge outfitted with fiber optic sensors [8.7]. The span is nearly 500 feet long, and crosses over the Assiniboine River in Canada. In the fiber optic sensor system used, the information gathered is transmitted through telephone lines to an office where an engineer can monitor the stresses and strains as they occur within the bridge. In addition to being outfitted with fiber optic sensors, the Headingley bridge utilizes advanced composite materials for reinforcement instead of steel. These advanced composites are 20% lighter and nearly 6 times stronger than steel, and it is expected that they will reduce maintenance costs and increase the service life of the bridge.

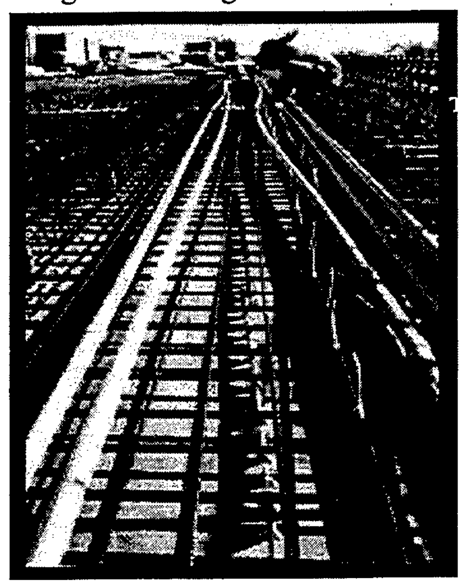

Figure 8.7 Conduits Housing the Fiber Optics of the Headingley Bridge [8.7].

8.4.3 Oklahoma

On a re-decking project for part of an Interstate-44 bridge in Oklahoma, a deck-heating system will be incorporated that draws heat up from the ground [8.25]. A major cause of corrosion in northern states is the use of salts on roadways to help eliminate ice. With a heated deck, salts would not be needed. The service life would be extended and the safety increased. This system is truly an example of smart technology – eventually the bridge will receive its information from the Oklahoma weather forecast system and act on that information without human intervention. For instance, it may be that if the weather system reports that a cold front will be moving through and precipitation conditions are likely, then the smart heating deck will begin operation and slowly heat-up before ice is formed.

8.4.4 Georgia

The Georgia Department of Transportation, working in collaboration with SMS, has instrumented four highway bridges including one newly constructed bridge with smart systems to monitor the health of each. The smart sensors developed by SMS record the critical strain in bridge elements. The sensors are networked and connected to a solar powered data acquisition and communication module. This module allows the bridge to store relevant information, send periodic reports, or warn if there is unusual activity. The information accumulated by these modules can be gathered as a specially equipped truck drives across the bridge and queries the system, or in other words, "the monitoring system will wake up, transmit its stored date up to the vehicle, then shut down, all without the vehicle having to stop" [8.26].

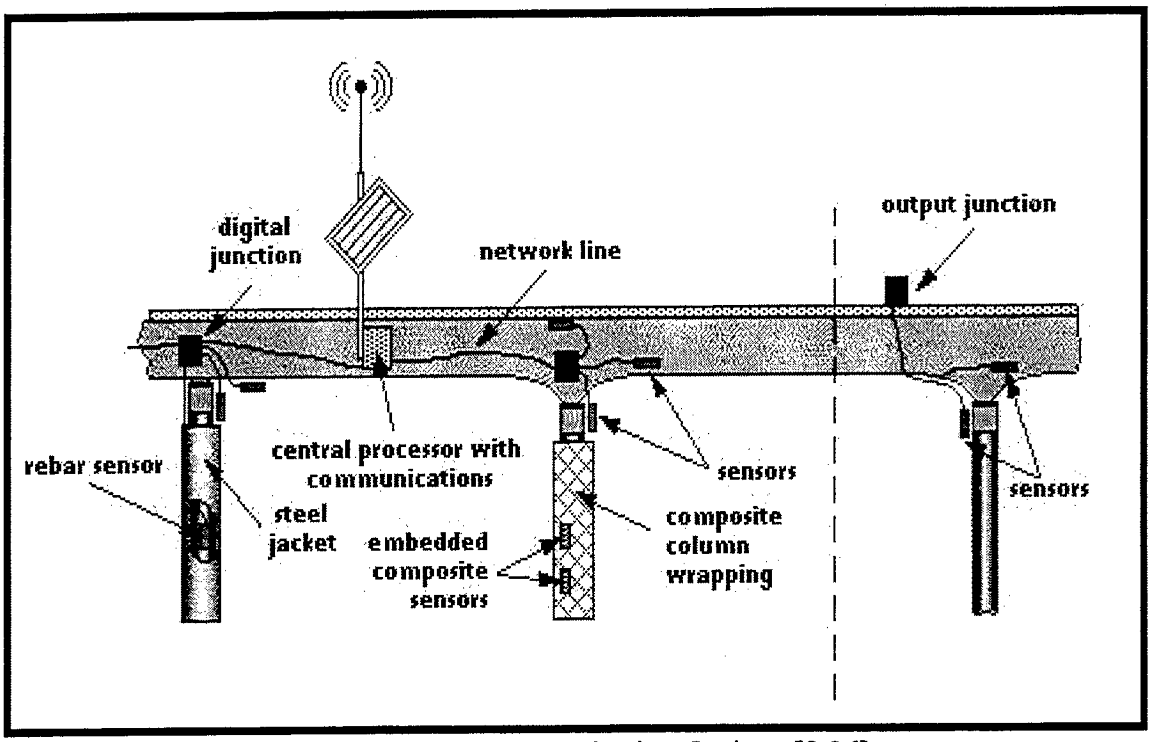

Figure 8.8 Bridge Monitoring Options [8.26].

8.4.5 Texas

The Center for Structural Control (CSC) at the University of Oklahoma has designed, installed, and tested an intelligent bridge that is part of I-35 in Texas [8.30]. The steel girder bridge utilizes a new hydraulic control technology known as a Semiactive Vibration Absorber (SAVA). This new technology aims to provide the performance achieved by active systems without the power requirements. When coupled with sensors and a small microcontroller, the CSC refer to the design as an intelligent stiffener for bridges (ISB). ISB, when attached to a bridge, act as muscles that can respond to stimuli and adjust the stiffness of the bridge as needed. For instance, by increasing the stiffness of the bridge during the passage of heavy trucks the ISB can reduce the peak stresses and thereby extend the life of the bridge.

8.5 The Future of Smart Bridges

In order for any new technology to make its way out of the laboratories and into the every-day use of the practicing engineer, it must prove itself of its benefits and practicality. Since smart technology is radically different than traditional civil engineering design methodologies, it must not only prove its benefits, but it must prove to be convincing as a consistent and coherent way of thinking. The common aim of all smart systems - to act and react in the same way that a biological system does - just may be the unifying bond necessary to pull smart technology together into a single, coherent new philosophy of engineering design.

8.5.1 Biomimetics Revealed

The source of inspiration for all the newest smart technology seems to be all too obvious. It is by no mistake that the fiber optics that are routed throughout a structure to monitor its health and warn if damage is present or imminent are given the nickname "glass nerves". After all, the biological nervous system does exactly the same thing – notifies that there is either current damage or possible future damage and appropriate action needs to be taken. When a biological creature incurs some type of wound, reparation comes from within the creature. In the case of the human being, that internal reparation comes in the form of platelets that travel in the blood, which clot together in the area of the wound until it has been sealed from its germ laden environment. Now compare this wound-healing process to the crack and corrosion "healing" process that takes place within a structure made of smart concrete, where a newly formed crack is mended by the adhesive released from within the concrete. There is really no conceptual difference. The shape memory alloy fibers placed within a concrete structure stiffen the structure when they are signaled to do so by the application of an electric current, much in the same way that a person's arm stiffens when the muscle fibers are signaled to flex by an electric impulse sent from the brain. There even exists a type of "smart paint" that changes color to notify when the steel to which it has been applied has been under a large amount of stress – in just the same way that a person's skin changes color, or bruises, after a large amount of pressure or stress.

The smart technology examined here could properly be called "biomimetics", a word used by Amato to describe a type of material science in which the scientists try to simulate natures own ingenuity in the creation of materials. As Amato puts it, the biomimetic credo is to "learn as much as possible about the material structures within organisms and to exploit that natural technology into a scientific or industrial context" [8.14]. But nature's genius has more to offer than the structure of the materials in her domain – the miracles of living organisms are there to be harnessed. If the current fiber optic sensors are found to be in some way lacking, maybe the answer to a better system lies in trying to closer simulate the biological nervous system that nature has spent so many billions of years perfecting. The same goes for the shape memory alloy actuator systems that we are looking so hopefully into. In no organism will you find muscles composed of titanium alloy, or any other metal for that matter. Instead you find long strands of protein – fast-twitch producing force with endurance and slow-twitch producing force with strength. If scientists can clone an exact replica of a sheep, they can surely grow custom strands of protein. With our scientific track record behind us and a biomimetic mindset ahead, it should not be impossible to foster large-scale sensors and actuators from the building blocks of life.

8.5.2 Integration is Key

As mentioned before, in order for a new technology to become accepted as the norm, it must prove itself to be wholly consistent and coherent. The more that its doctrines prove to be useful tools, the more widely accepted it becomes. As smart technology becomes more and more widely accepted as a plausible design philosophy, the more frequently its individual doctrines will appear together, coexistent in the designs of engineers. Eventually, they may fuse together, forming a smart structure whose individual components are indistinguishable from one another. This fusion seems almost imminent even in theory.

Again we look to biomimetics for some insight. It would be quite difficult, if not impossible to position the nervous system of a human inside their body after the fact. Instead, the nervous system grows with the body, in what seems to be a never-ending series of branches, until every square inch is populated by some type of nerve. The fiber optic system that we have been correlating to the nervous system should have a similar means for an easy distribution. These fiber optic sensors operate by means a traveling light and its disturbance. We have the ability to grow crystals, and some crystals have the ability to carry light, so we therefore have at least some potential to "grow" a structural nervous system. It is understood that current technology may not lend a crystal with clear enough light carrying abilities, or even the ability itself to direct the growth of such crystal, but it has at least some potential that is worth considering further.

Such crystals could also be fabricated hollow, with minute continuous voids, and injected with the adhesive or anticorrosive chemicals found in the fibers of smart concrete. The light carrying properties could possibly be gauged and corrected, if altered at all. If the entire crystal neural system had grown incorporating a network of microscopic tunnels, then the adhesive or anticorrosive chemicals could be supplied indefinitely. Such a crystal-optic system could grow to create an all-encompassing system that not only detects and constantly monitors its health in the widest distributed area, but also takes action to counteract cracking and corrosion.

8.6 The Need for Smart Bridges

The ultimate extent to which smart technology is utilized in future bridge design depends on the technology having positive engineering, safety, and economic benefits. If the benefits are not currently obvious, they must be in sight within a reachable distance. A smart bridge design that has the same service life as its dumb counterpart, yet costs twice as much and has no other obvious benefits would be a hindrance to society, not a service. But with the infrastructure in the state it is, the scientific community must push for a change. No technology is cheapest when it is first discovered–instead it is at that point that it is most expensive and reducing as more is learned and it becomes more widely applied.

8.6.1 The Status of Today's Bridges

Each year the United States spends at least 5 billion dollars for highway bridge design, construction, replacement, and rehabilitation, and yet approximately 35% of the nations 575,000 bridges are classified as deficient [8.22]. This deficiency does not necessarily mean that the bridge is unsafe, just that it can no longer carry the loads it was originally designed to carry. Many bridges are rapidly deteriorating under the weight of increased traffic loads, extreme age, and severe weather. Furthermore, rehabilitation and replacement plans cannot keep up with the rate at which the bridges are aging, as the cost associated with repairing all backlogged bridge deficiencies increases exponentially each year, as the costs increase the more the repairs are delayed.

8.6.2 The Cost of Repair and Maintenance

It has been estimated that somewhere between 75 billion to 100 billion dollars is needed to bring all deficient bridges in the United States up to acceptable standards [8.23]. The cost to simply maintain overall bridge conditions is estimated at 5.2 billion dollars every year, and an addition 8.2 billion dollars are required annually to handle accruing deficiencies – this is the equivalent of rehabilitating or replacing 12000 bridges each year [8.22]. Yet the sum of all funding from federal, state, and local government is about 5 million dollars. With these figures in hand, it is easy to see that bridge deficiencies will not cease to grow in number any time soon.

8.6.3 Looking Toward the Future

With the adoption of this new philosophy known as smart technology, these repair and maintenance costs may be curbed. It may be true that the costs of smart bridges will initially increase the figures illustrated here, but in the long run the technology will become more economically beneficial the more it is used. Once the technology surpasses this economic barrier, it will not only pay for itself, it will allow for the regeneration and possible retrofitting of the deficient bridges that currently plague our country. Smart bridges would be able to notify that a repair is needed *before* that it becomes an economic burden.

8.7 Conclusion

Traditionally, civil engineers have designed structures for events that are not likely to occur. Their designs have been based on the certainties and uncertainties that a severe event will occur, which will test the limits of their creation. Mass is always added to a final structural design to prepare it for the worst-case scenario. But such methodology requires more natural resources, more energy, and more time, and yet since the absolute worst-case cannot be predicted, it does not guarantee that the structure will not fail.

By incorporating smart technology instead of simply adding mass, an engineer can enable a bridge to react and respond to host of circumstances that may be thrown its way. Sensors can be incorporated to notify the engineer that repair is needed, before that repair becomes costly. Smart concrete can be used that would automatically mend any cracks before they become a problem, and stop corrosion before it begins. Actuator systems could be incorporated to cancel out the damaging vibrations that cause so many fatigue problems. Most important of all, all of this can be incorporated together, so that the bridge of tomorrow can sense and repair its own damage, and through increased service life and decreased reliance on natural resources, be just as economically feasible as any bridge found today.

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9. PERFORMANCE OF FLORIDA'S LONG SPAN BRIDGES

9.1 Introduction

Problems associated with bridge superstructures are well-known, e.g. expansion joints, and documented in books, e.g. Ref 9.1. However, Florida's bridges in the 200-600 ft range are relatively new averaging between 6-14 years (see Section 2.9). Consequently, an attempt was made to determine the specific types of problems that these newer structures had been encountering.

The *modus operandi* was to conduct a detailed search of the Florida Department of Transportation's bridge inspection and maintenance files. The proximity of USF's campus to FDOT's Maintenance Offices for Districts 1 and 7 made them the logical choice for this study. These two districts have a total of fourteen bridges spanning between 200 and 600 feet in their jurisdiction all of which were examined for this analysis. This constitutes about 1/7th of the total number of bridges in this span range in the state. The bridges were girder or box type made of steel or prestressed concrete. On an average, inspections took place at 2-year intervals regardless of the environment, whether extremely or moderately corrosive, in which the bridge was located.

Using Microsoft Office software, compact data files were created for each of the fourteen bridges. These contained structural information and complete details of the bridge maintenance inspection findings. In order to carry out comparable analyses and evaluate maintenance problems, descriptive information and recognized deficiencies in the FDOT maintenance files had to be first categorized into specific types of deficiency, e.g. cracking, misalignment, spalling. A database was then constructed using Microsoft Access software to manage and present the collected data.

A brief summary on the common problems discovered is presented here. For convenience, the structures have been categorized into three groups - two relating to steel bridges and the third to prestressed concrete. Background information on the bridges examined is given in Section 9.2. Sections 9.3 and 9.4 relate to the performance of the steel bridges while Section 9.5 considers those made of prestressed concrete. The overall conclusions from the analysis are summarized in Section 9.6. Two appendices provide supplementary information. Appendix A is a listing of the queries created in Microsoft Access database and provides details of deficiencies identified during each inspection. Appendix B provides supporting sketches and photographs of some of the deterioration that was recorded.

9.2 Description of Bridges

Information on the 14 bridges examined is summarized in Tables 9.1-9.3. Tables 9.1 and 9.2 relate to steel plate girder and box girder bridges respectively with Table 9.3 providing information on the prestressed concrete bridges. Each table follows the same format; it contains details of the bridge number, the year it was built, its location, the classification of the corrosion environment, its maximum span and skew.

FDOT categorizes environments as "slightly corrosive", "moderately corrosive" or "extremely corrosive" taking into consideration pH, resistivity, and sulfate/chloride concentration levels. Environments are classified as extremely corrosive if the chloride concentration exceeds 2000 parts per million. These are identified as "severe" in the tables. Moderate environments are those where the chloride concentration falls between 500-2000 ppm. Seven of the fourteen bridges are located in environments classified as extremely corrosive by FDOT. The remaining seven are in moderate environments. Interestingly, six of the ten steel bridges are in severe environments.

Table 9.1 Details of Steel Plate Girder Bridges.

Bridge Number	**Year Built**	**Location**	**Corrosion Environment**	**Max. Span ft**	**Skew deg**
100296	1976	I-275 NB ramp to SR-60 over SR-60, located 0.2 miles North of I-275	Severe	233.3	70
150149	1980	I-275 SB over 31st Street South, located 4.3 [km] North of SR-682	Severe	232.9	99
100354	1981	21st Ave over I-75, located 4.4 miles North of Manatee CO	Moderate	223.1	59
130090	1981	I-275 NB connector "A" over I-75, located 5.3 [km] North of US-301 (SR-670)	Moderate	244.0	0
150186	1984	I-275 SB ramp to SR-682 WB, located at intersection of US-19 and SR-682	Severe	229.0	99
150187	1984	SR-682 EB ramp to I-275 NB, located 4.3 [km] North of SR-682	Severe	219.0	99
150190	1987	I-275 SB over I-275 NB / US-19 SB, located 0.2 miles North of SR-682	Severe	229.0	99

All structures are continuous and support concrete decks

Table 9.2 Details of Steel Box Girder Bridges.

Bridge Number	Year Built	Location	Corrosion Environment	Max. Span ft	Skew deg
100491	1985	I-75 NB collector ramp "E" over ramp "G", located 1.7 miles South of SR-60	Moderate	264.1	62
100498	1987	I-75 NB collector ramp "E" ramp "B", located 1.7 miles South of SR-60	Moderate	283.0	0
150204	1990	US-19 over Ulmerton Road, located 7 [km] South of SR-60	Severe	228.0	32

Only the last bridge (span 228 ft) is continuous

Table 9.3 Details of Prestressed Concrete Bridges.

Bridge Number	Year Built	Location	Corrosion Environment	Max. Span ft	Skew deg
130112	1981	Connector AD over I-75 and I-275, located 1.0 [km] East of US-41	Moderate	251.0	0
150189	1986	I-275 Skyway Bridge, located 5 miles South of Pinellas Bayway	Severe	540.0	0
150224	1995	4th Street exit ramp from I-275 SB, located 0.5 mile South of Howard Franklin Bridge	Moderate	208.0	55
100585	1997	Gandy Bridge WB, over Tampa Bay	Moderate	234.3	0

All bridges are continuous

Several of the bridges are ramp structures. The largest span is 540 ft and the smallest, 208 ft (Table 9.3). Thirteen of the fourteen structures (92.8%) have spans below 300 ft. This agrees well with the statewide average of 91.5% (see Table 2.13).

Inspection of Tables 9.1-9.3 indicates that the oldest bridge in the group was built in 1976, the youngest in 1997. The average age of the steel bridges is 14.5 years, comparable to the statewide average of 14 years (see Table 2.12). Those for the prestressed concrete bridges average 8.3 years somewhat greater than the statewide average of 6 years. Thus, the bridge sample examined in many ways is representative of bridges statewide.

9.3 Steel Plate Girder Bridges

The seven plate girder bridges examined ranged between 9-17 years at the time of the last inspection. The principal findings are summarized in Figs 9.1-9.2. Fig. 9.1 provides information on *components* that were observed to deteriorate. Fig. 9.2 provides information on *type of deficiency* that was detected. Supplementary information is provided in Tables 9.4-9.6. A complete catalogue of the deficiencies may be found in Table 7A of Appendix A.

The components that were found to have problems were the **deck slab, expansion joints, approach slabs, main girders** and **bearings**. Some deficiencies were also detected in *barrier walls, bracing* and *diaphragms*. The relative proportions of these defects from the 103 items listed in the maintenance reports are summarized in Table 9.4.

Table 9.4 Deficient Plate Girder Components.

Deficient Component	Percent of deficiencies [%]
Deck slab	29.1
Expansion joints	21.4
Approach slab	13.6
Main girders	12.6
Bearings	11.6
Barrier walls	4.9
Bracing	3.9
Diaphragms	2.9
Total	100%

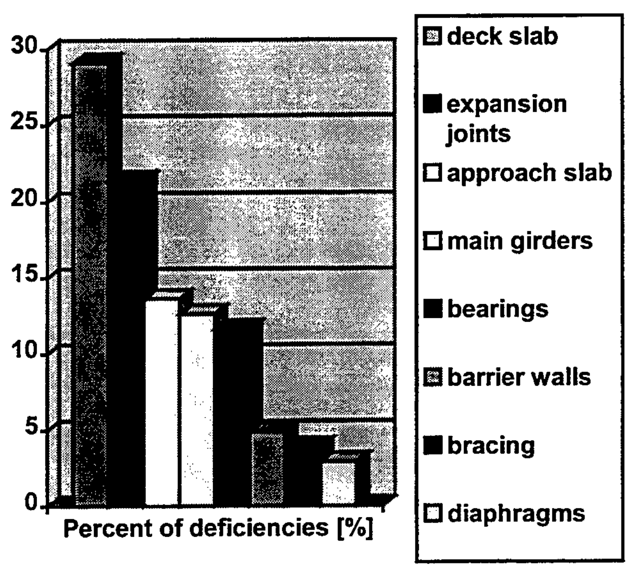


Figure 9.1 Deficient Components in Plate Girder Bridges Listed in Table 9.1.

Surprisingly, the largest portion of the deficiencies relates to the deck slab and the approach slabs rather than the expansion joints. Specific details are provided later. Problems with expansion joints were also persistent; six of the seven bridges examined developed some deficiency, primarily relating to looseness or deterioration of the sealant material.

Bearings were less problematic. Minor corrosion and missing bolts were detected in four of the seven bridges even though bearings constituted over 11% of all deficiencies reported. Serious misalignment problem was detected in one of the seven analyzed bridges (see Figure 7B in Appendix B for sketch) probably resulting from fabrication defects of the curved girders. Overall, steel girders did not seem to pose severe maintenance problem despite the location of several of the bridges in extremely corrosive environment (see Table 9.1). After nine years of service, only minor corrosion problems were observed in five of seven analyzed bridges.

Supplementary information relating to the type of deficiency observed is listed in Table 9.5. Deck slab problems corresponded to **concrete cracking** and **spalling**. The remaining three most frequent deficiencies were **steel corrosion**, **paint peeling** of the steel components of the superstructure, and **expansion joints deterioration**.

Table 9.5 Deficiency Types in Plate Girder Bridges.

Deficiency Type	Percent of deficiency type [%]
Concrete cracks	36.8
Concrete spalls	15.6
Steel corrosion	11.6
Paint peeling off	11.6
Joints deterioration	10.7

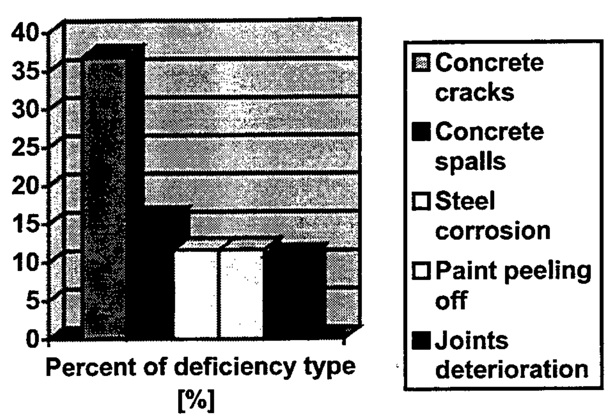


Figure 9.2 Deficiency Types in Plate Girder Bridges listed in Table 9.1.

Cracking of the concrete slab was a very common problem. Transverse crack patterns were most frequent although longitudinal and diagonal cracks were observed in five of seven bridges analyzed (see Table 1A in Appendix A). Overall, over 68% of detected concrete slab cracking deficiencies were minor Class I problems [crackwidth under 1/64 in.], 26% were Class II and III [crackwidth between 1/64 in. to 1/16 in.] and less than 6% were Class V [crackwidth more than 1/8 in.]. Several example sketches of the observed cracking problems may be found in Appendix B. Table 9.6 gives a breakdown of the various deficiencies.

Table 9.6 Breakdown of Deficiency Type in Plate Girder Bridges.

Deficiency Type	Number of detected problems	Percent of detected problems [%]
Crack, Class I	26	25.2
Crack, Class II	7	6.8
Crack, Class III	3	2.9
Crack, Class V	2	1.9
Spall	16	15.6
Misalignment	4	3.9
Steel corrosion, minor	7	6.8
Steel corrosion, moderate	5	4.9
Deflection due to heavy traffic	1	1
Joint deterioration	11	10.7
Loose, missing or sheared anchor bolt	8	7.7
Asphalt Overlay	1	1
Paint peeling off	12	11.6
Total	**103**	**100%**

Spalling of the concrete deck was the second most frequent problem observed. Spalls were detected in six of the seven bridges examined and were usually the consequence of the neglect of cracking deficiency problems (see Figure 2B in Appendix B for examples of the concrete deck spalls in the beginning stage and in the more advanced stages with exposed rebars). Over two thirds of detected spall deficiencies developed along the loose elastomeric expansion joints (see Figure 3B in Appendix B for example pictures of this type of spalling).

Minor steel corrosion was the consequence of paint peeling off. This was observed in girders, bearings, bracing elements and diaphragms. Moderate corrosion was confined primarily to the girders, in one instance resulting from impact damage. As mentioned earlier, the lack of evidence of significant corrosion after seventeen years of service is encouraging (see Appendix A for details).

9.4 Steel Box-Girder Bridges

The three box-girder bridges examined varied between 6-11 years at the time of the last inspection. The analysis of the maintenance records is presented in the same format as that for the plate girder bridges. Information on deficient components is presented in Fig. 9.3 followed by details on the type of deficiency in Fig. 9.4. Additional information is provided in Tables 9.7-9.9 while a complete listing of the deficiencies is summarized in Table 8A of Appendix A.

The major components that were found to have deficiencies were the **deck slab, main girders, approach slab, expansion joints** and **bearings**. In addition, roadway alignment problems were also reported. The relative proportions of these defects from the 20 items listed in Appendix A are summarized in Table 9.7 and shown graphically in Fig. 9.3.

Table 9.7 Deficient Box-Girder Bridge Components.

Deficient Component	Percent of deficiencies [%]
Deck slab	35.0
Main girders	20.0
Approach slab	15.0
Expansion joints	15.0
Bearings	10.0
Roadway alignment	5.0
Total	100%

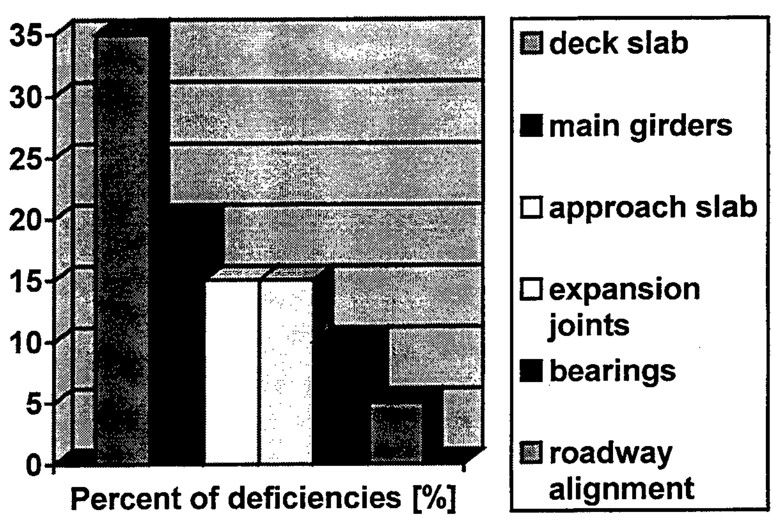


Figure 9.3 Deficient Components in Steel Box Girder Bridges in Table 9.2.

As for plate girder bridges, the greatest number of problems were associated with the deck slab. Girders registered the second largest number of deficiencies mainly the result of faulty construction. Complete listing of deficiencies in the various components in each bridge examined may be found in Table 8A in Appendix A.

Table 9.8 summarizes the principal types of deficiencies reported. These are re-plotted in Fig. 9.4. As for the plate girder bridges, cracking of the deck slab and spalling constituted the two largest categories. Joint deterioration was next. Corrosion did not feature in this list possibly because of the relative newness of the bridge in the most severe environment - the inspection records were after six years.

Table 9.8 Deficiency Types in Steel Box-Girder Bridges.

Deficiency Type	Percent of deficiency type [%]
Concrete cracks	45.0
Concrete spalls	10.0
Joints deterioration	10.0

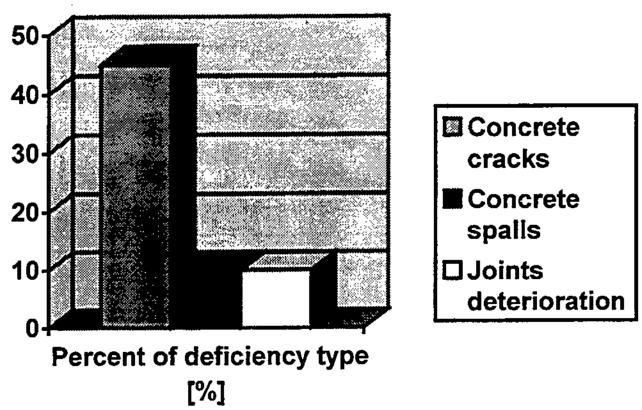


Figure 9.4 Deficiency Types in Steel Box Girder Bridges.

Additional information on the various types of deficiencies is summarized in Table 9.9. Of the 45% of the cracks reported in Table 9.8, 10% are Class I [crackwidth under 1/64 in.], 25% are Class II [crackwidth between 1/64 in. and 1/32 in.]and 10% are Class III [crackwidth between 1/32 in. and 1/16 in.]. Expansion joint deterioration problems due to deteriorated material were also reported. Steel corrosion, loose or sheared anchor bolts and new construction defects made up some of the remaining types of deficiency that were observed.

Table 9.9 Breakdown of Deficiency Types in Steel Box-Girder Bridges.

Deficiency Type	Number of detected problems	Percent of detected problems [%]
Crack, Class I	2	10.0
Crack, Class II	5	25.0
Crack, Class III	2	10.0
Spall	2	10.0
Misalignment	1	5.0
Joint deterioration	2	10.0
Steel corrosion, minor	1	5.0
Loose, missing or sheared anchor bolts	1	5.0
Missing access hole mounting	1	5.0
Roadway rise at approach slab	1	5.0
New construction defect - to thin coat	1	5.0
New construction defect - box girder	1	5.0
Total	**20**	**100%**

9.5 Prestressed Concrete Bridges

The four prestressed bridges examined varied between 0-16 years at the time of the last inspection. The results of the inspection records are presented in the same format as those for the steel bridges. Information on deficient components is first presented followed by a broad division of deficiency types and their detailed breakdown. As before, complete information may be found in the appendices.

Table 9.10 and Fig. 9.5 summarize information on the main components that were found to be problematic in prestressed concrete bridges. While problems for deck slabs, expansion joints and bearings were similar to those in steel bridges, there were additional problems related to cracking of the concrete girders either when new or after inspection.

Table 9.10 Deficient Components in Prestressed Bridges Listed in Table 9.3.

Deficient Component	Percent of deficiencies [%]
Deck slab	33.3
Main girders	33.3
Expansion joints	18.2
Bearings	15.2
Total	100%

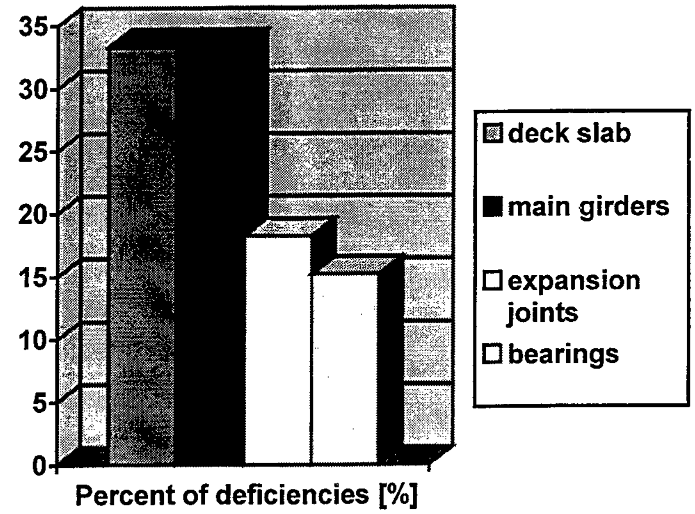


Figure 9.5 Deficient Components in Prestressed Concrete Bridges in Table 9.3.

Table 9.11 lists the type of deficiencies that were most prevalent. Concrete cracking accounted for 54.6% of the defects followed by joint deterioration that accounted for 21.2%. Spalling and bearing corrosion accounted for 9.1% of the total deficiency. The information is also plotted in Fig. 9.6.

Table 9.11 Deficiency Types in Prestressed Bridges Listed in Table 9.3.

Deficiency Type	Percent of deficiency type [%]
Concrete cracks	54.6
Joints deterioration	21.2
Concrete spalls	9.1
Bearing corrosion	9.1

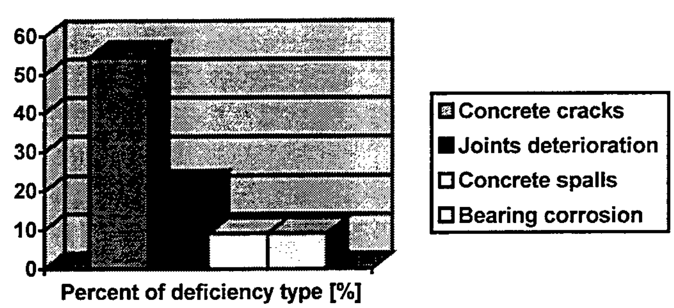


Figure 9.6 Deficiency Types in Prestressed Concrete Bridges in Table 9.3.

A breakdown of the variety of deficiencies in Table 9.11 is presented in Table 9.12. Even though cracking constituted the biggest problem, the extent of cracking in the concrete bridges was less severe. Over 78% of detected cracking deficiencies were minor Class I problems [crackwidth below1/64 in.], 17% were Class II [crackwidth between 1/64 in. and 1/32 in.], and only 5% were Class IV [crackwidth between 1/16 in. and 1/8 in.]. Additionally, only two of four bridges (50%) in this group developed this type of deficiency compared to 100% in the bridges made of steel.

Table 9.12 Breakdown in Deficiency Types in Prestressed Bridges Listed in Table 9.3.

Deficiency Type	Number of detected problems	Percent of detected problems [%]
Crack, Class I	14	42.5
Crack, Class II	3	9.1
Crack, Class IV	1	3.0
Spall	3	9.1
Bearing corrosion,	3	9.1
Joint deterioration	7	21.2
Misalignment	1	3.0
Leaks into the concrete box	1	3.0
Total	**33**	**100%**

9.6 Conclusions

This chapter provides a very brief summary of information obtained on the performance of 14 representative long span bridges in the jurisdiction of Districts 1 and 7. Fifty percent of the bridges were steel plate girder bridges, 21% were steel box girder bridges and the rest were made of prestressed concrete. The age of the bridges varied from zero at the time of inspection to 17 years.

The results of the survey indicate that cracking of the concrete slab was the most common problem. In plate girder bridges, the cracks were mostly transverse though instances of longitudinal and diagonal cracking were also observed. Neglect of cracking led to spalling of the concrete and deterioration of the reinforcement. This problem appears to be one that can be readily rectified by design.

There was very limited corrosion in the steel bridges despite their exposure to severe environments. In contrast, prestressed concrete girders developed cracks prior and during service. This problem warrants further investigation. Apart from cracking of the deck slab, there were well-known failures of expansion joints and at bearings.

References

9.1 Silano, L. (1992). Bridge Inspection and Rehabilitation - A Practical Guide, John Wiley InterScience, New York, NY.

10. EXAMPLES OF INNOVATIVE BRIDGES WORLDWIDE

10.1 Introduction

A literature search was conducted to obtain information on innovative bridges constructed world-wide. This chapter provides brief descriptions of a selection of 21 bridges that highlight some of the more recent developments relating to new concepts, materials or construction methodology.

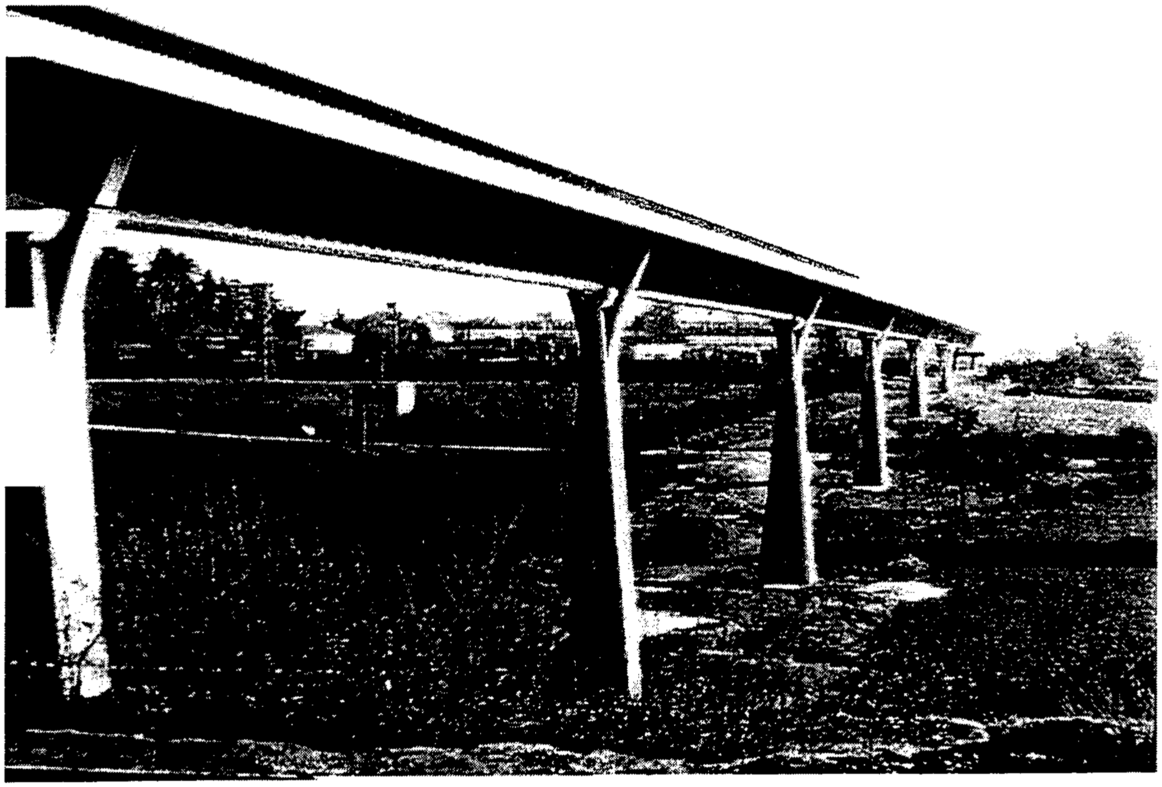
A two-page format is used to describe each of the bridges; the first page contains a photo along with pertinent information such as type, location, span, year of construction. The second page contains technical information and references where additional information may be found. Section 10.2 focuses on bridges defining new concepts. Section 10.3 identifies bridges utilizing old concepts but new materials while Section 10.4 relates to innovative construction.

10.2 New Concepts

This section describes ten innovative bridges built in England, France, Japan, Spain and the United States since 1987 and covers pages 10.2 to 10.25. Two of the bridges are currently under construction. The examples presented are not solely confined to highway bridges nor to spans in the 61-183 m range since the goal is to present the concept.

Among the most promising concepts for steel bridges in the 61-183 m span range are those utilizing *corrugated webs*. Four examples illustrating both plate girder and box girder applications of this type of bridge are presented. Two of these are from France (Maupre Viaduct and the Dole Bridge) and the remaining two from Japan (Shinkai Bridge and Hontani Bridge). Another promising concept suitable for prestressed structures is the *extradosed bridge*. Two examples of such bridges recently constructed in Japan - the Odawara Blueway Bridge and the Natorigawa Bridge - are described. Six additional examples of elegant but perhaps more costly structures are also included. These are a *stress-ribbon bridge* (Shiosai Bridge, Japan), a *cable stayed bridge with a single inclined tower* (Alamillo Bridge, Spain and the Sun-Marine Bridge Japan), *a composite space truss post-tensioned bridge* (Roize Bridge, France), *tubular arch bridge* (Antrenas Bridge, France) and a *steel arch bascule bridge* (Tyne Millenium Footbridge, England).

Maupre Viaduct



Location	Charolles, France
Type	Corrugated triangular box girder
Span	54 m (177.2 ft)
Owner	Ministri de Transports
Designer	Campenon Bernard Sge
Year	1987

Maupre Viaduct

The Maupre Viaduct is the first bridge having a triangular cross-section using webs made of corrugated steel. The corrugated webs are 8 mm thick, 3.1 m high, welded at the bottom to a concrete-filled 610 mm diameter steel pipe. Metal forms for the concrete slab also serve to close the structural triangle. Steel angles welded to the corrugated webs facilitate connection to the metal form.

The structure was assembled in segments on a curved platform to introduce stresses in the deck that counter top slab shrinkage effects. The segments were welded, the concrete slab cast and prestressing force applied. A complete span was then pushed in place. Additional details may be found in the reference listed. This construction type is protected by a registered patent.

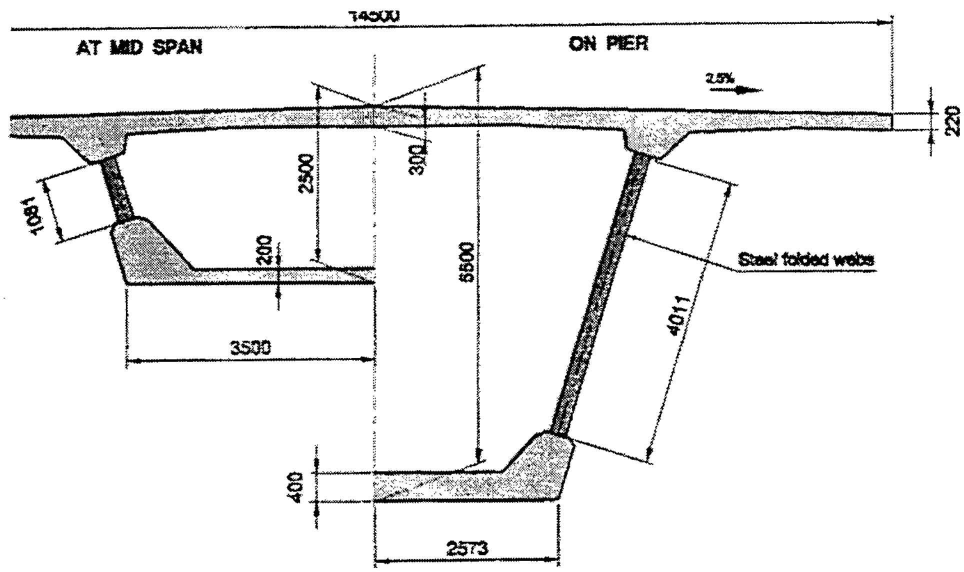
Reference

Campenon Bernard Sge (No Date). *The Charolles Bridge*, Rueil-Malmaison, France.

Dole Bridge

Location	Dole, France
Type	Corrugated web box girder
Span	80 m (262.4 ft)
Owner	Ville de Dole
Designer	Campenon Barnard Sge
Year	1995

Dole Bridge



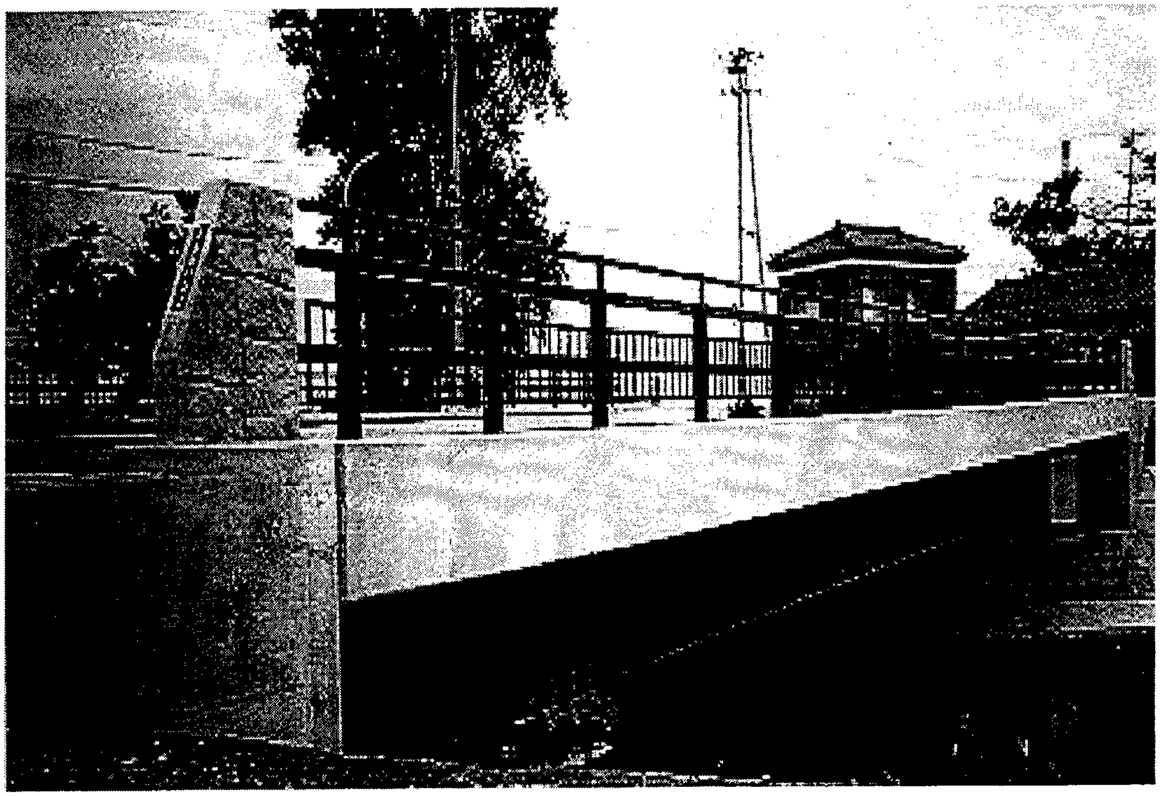
The Dole bridge is the first corrugated web haunched box girder bridge to be built by the cantilever method. It is a continuous seven span box girder bridge with variable depth, maximum at the pier and minimum at the mid span. Corrugated web thickness varies from 8 mm to 12 mm, thickness is maximum where prestressing cables are deflected and minimum at the abutments. Low axial rigidity of the corrugated webs not only introduces prestress effectively but also results in a lighter and more efficient section.

The Dole bridge is prestressed by three families of tendons, namely cantilever, continuous and external. Cantilever tendons were anchored to the top slab and tensioned after completion of each pair of symmetrical segments. After the cantilevers were assembled they were joined by continuous tendons to balance any tension induced by thermal effects. The whole structure is prestressed by external cables and the tendon profile is achieved using two concrete diaphragms per web. Additional details may be found in Campenon Bernard Sge (1995). This construction type is protected by a registered patent.

Reference

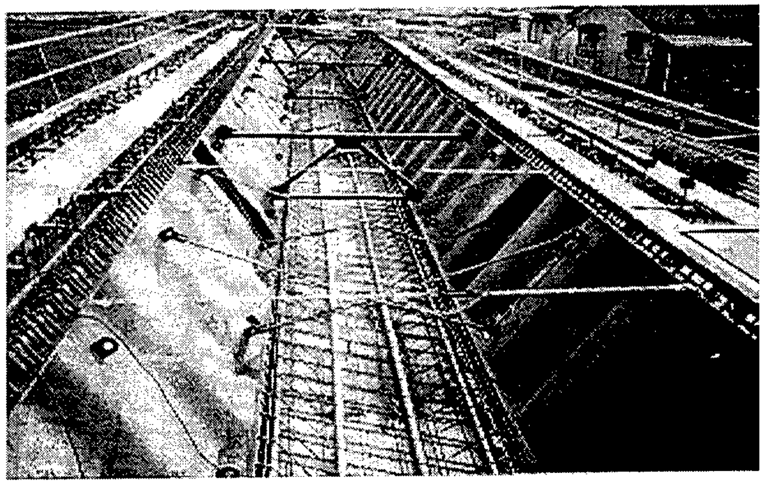
Campenon Bernard Sge (1995). *The Dole Bridge*, Rueil-Malmaison, France.

Shinkai Bridge



Location	Niigata City, Japan
Type	Prestressed corrugated box girder
Span	30 m (98.4 ft)
Owner	Niigata Prefecture
Consultants	Pacific Engineering Co., Ltd.
Year	1993

Shinkai Bridge



The Shinkai Bridge is a composite prestressed concrete box girder bridge in which the webs are made of 9 mm thick corrugated steel. Stud shear connectors are welded to the corrugated webs at their top and bottom to connect them to the concrete flanges and ensure composite action. Prestressing is provided by internal tendons embedded in the concrete and by external tendons that are placed outside. Special consideration was given to permit easy replacement of the external tendons in the future.

The Shinkai Bridge is the first bridge in Japan to combine the use of corrugated webs and external prestressing. World-wide, the *Maupre Viaduct* in France was the first such bridge to be built. The use of corrugated webs not only led to a lighter structure, but also allowed more efficient distribution of prestressing forces to the concrete slab. From an aesthetic stand point, corrugations enhance the appearance of the bridge.

References

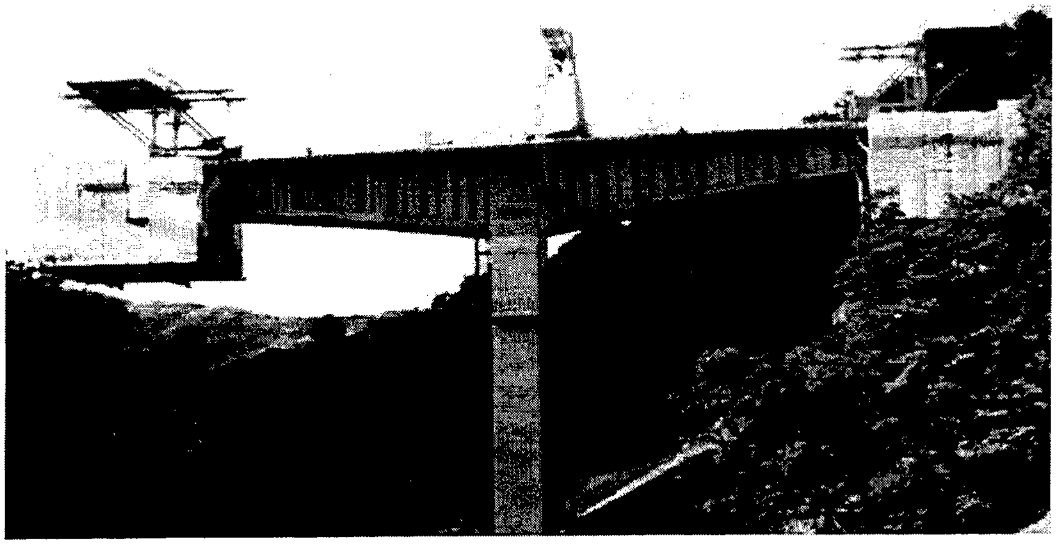
Takao Naito and Masaaki Hattori (1994). "Prestressed Concrete Bridge Using Corrugated Steel Webs Shinkai Bridge," *Japan Prestressed Concrete Engineering Association*, National Report, pp. 101-104.

Jacques Combault (1988). "The Maupre Viaduct Near Charolles, France," *Proceedings of 1988 National Steel Construction Conference, June.*

Hontani Bridge

Location	Gifu Prefecture, Japan
Type	Corrugated box girder
Span	97.2 m (318.9 ft)
Owner	Japan Highway Corporation
Year	1998

Hontani Bridge



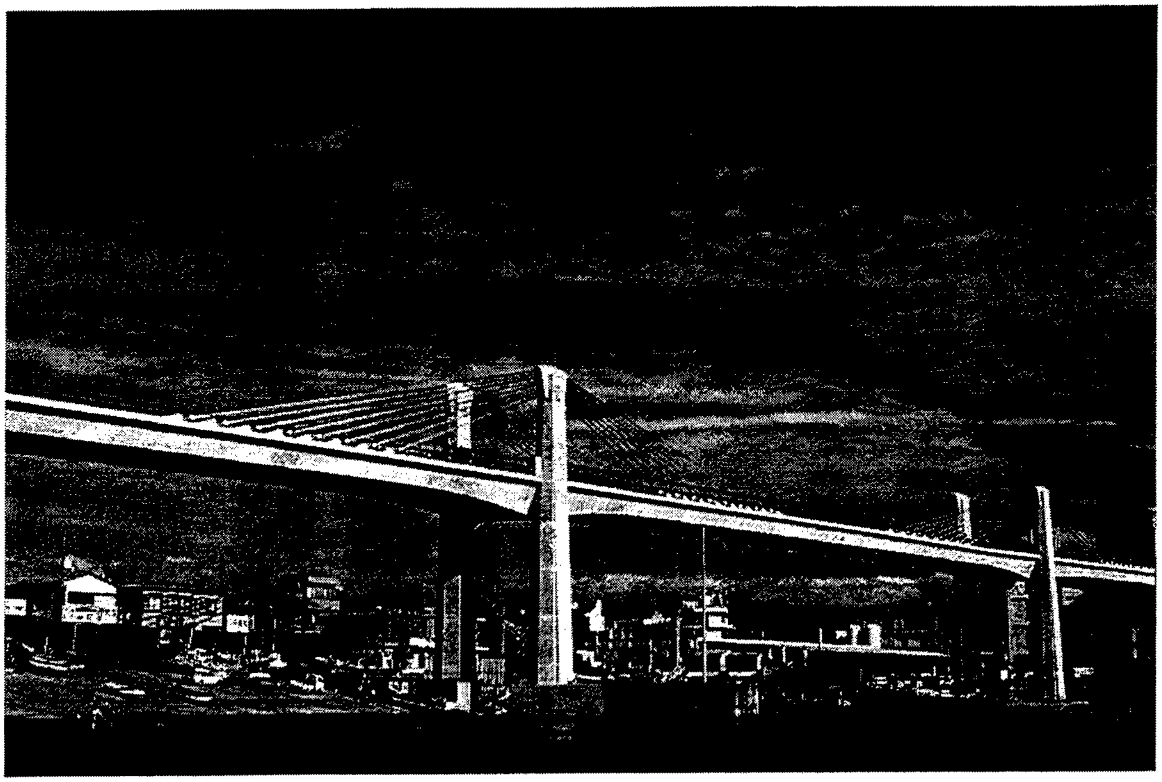
The 198m long Hontani Bridge is currently under construction and is expected to be completed in 1998. The design of this composite prestressed structure is very similar to that of Shinkai Bridge. The corrugated webs are 14 mm thick at the supports and vary from 9-14 mm at mid span. Both internal and external prestressing tendons are used.

The depth of the bridge varies with the span. The maximum depth is 6.4 m at the supports and 2.5 m at the mid-span. The superstructure cost is estimated as ¥940 m (about $8.15 m). The overall appearance is very similar to the Dole Bridge.

Reference

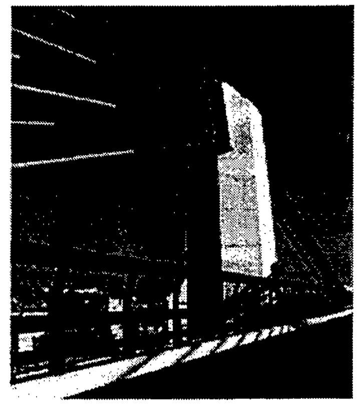
Tanka, K. (1998). *P.S Corporation,* Tokyo, Private Communication, Japan, May 15.

Odawara Blueway Bridge



Location	Odawara, Japan
Type	Extradosed prestressed bridge
Span	122 m (400.2 ft)
Owner	Japan Highway Corporation
Year	1994

Odawara Blueway Bridge



The Odawara Blueway Bridge is an "extradosed" prestressed concrete bridge. In this bridge type, prestressing tendons are placed *outside* the girder to increase eccentricity. As a result, much higher span to depth ratios are achievable. The span to depth ratio of the Odawara bridge is 30 compared, for example, to 22 for the Moore Haven Bridge, the longest splice girder bridge currently under construction in Florida. Extradosed bridges *resemble* cable stayed structures and display some similar characteristics. However, their structural behavior is closer to that of ordinary girder bridges.

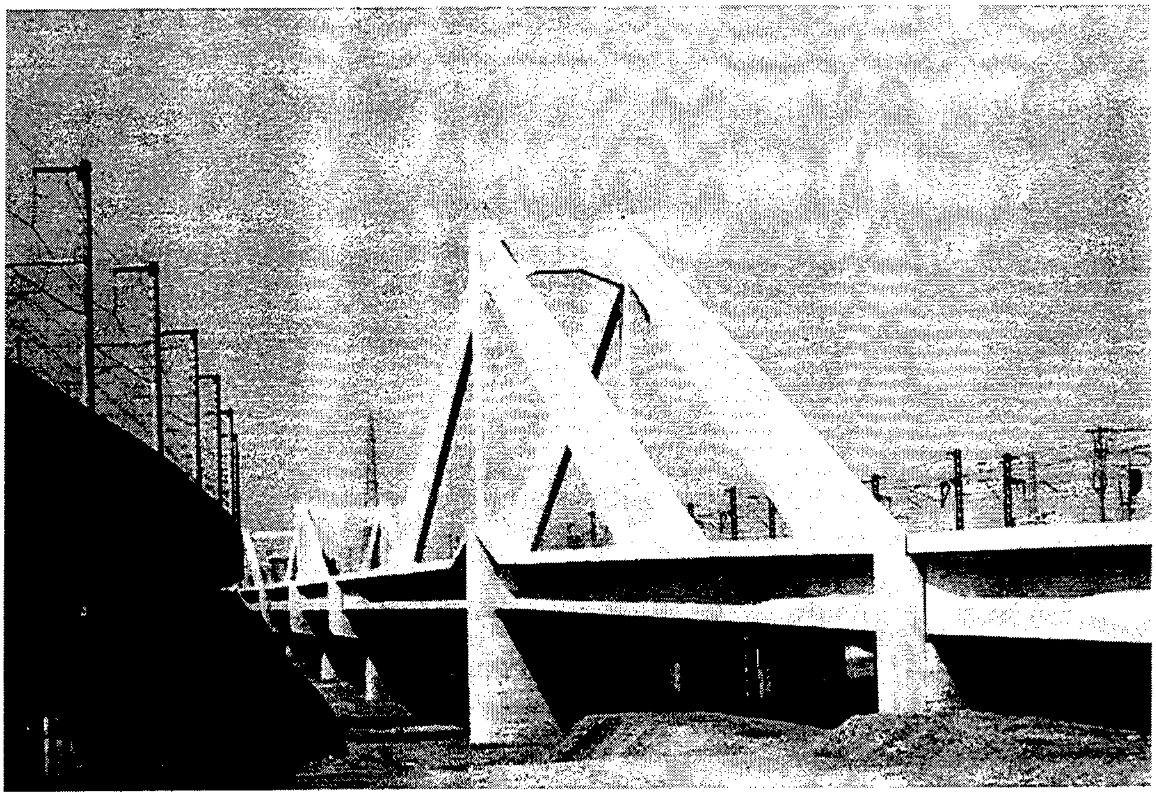
The Odwara bridge differs from previous extradosed prestressed bridges constructed, e.g., Barton Creek Bridge, CA, Gantor Bridge, Switzerland, in that the stay cables are not covered by concrete. Consequently, the dead load of the structure is smaller. Compared to cable stayed structures, the ratio of the tower height to span length in the Odawara bridge is considerably lower - 1:12 vs 1:5. The resulting shallower slope of the cables attract a much smaller component of the live load. The design also limits the maximum cable stress to $0.6f_{pu}$. As changes in live load stress are insignificant, fatigue considerations become less important. The stay cables are taken over a saddle structure fabricated at the top of the tower, avoiding the need for anchoring stay cables in the tower and leading to a less congested/economical solution.

The high span to depth ratio in extradosed prestressed bridges make it an economical solution for spans between 100-200m bridges that fall between the limits for girder and cable-stayed bridges.

Reference

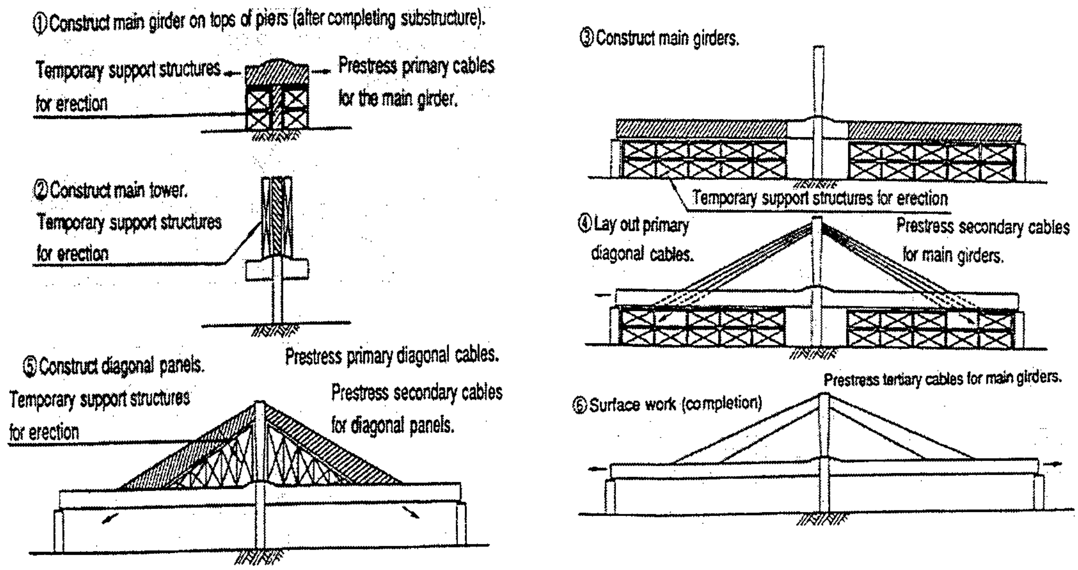
Brochure on Odawara Blueway Bridge (no date), *Japan Bridge & Structures Institute Inc.*, Consulting Engineers, Tokyo, Japan.

Natorigawa Bridge



Location	Japan
Type	Panel Stayed Bridge
Span	108.6 m (356.2 ft)
Owner	East Japan Railway Company
Year	1996

Natorigawa Bridge



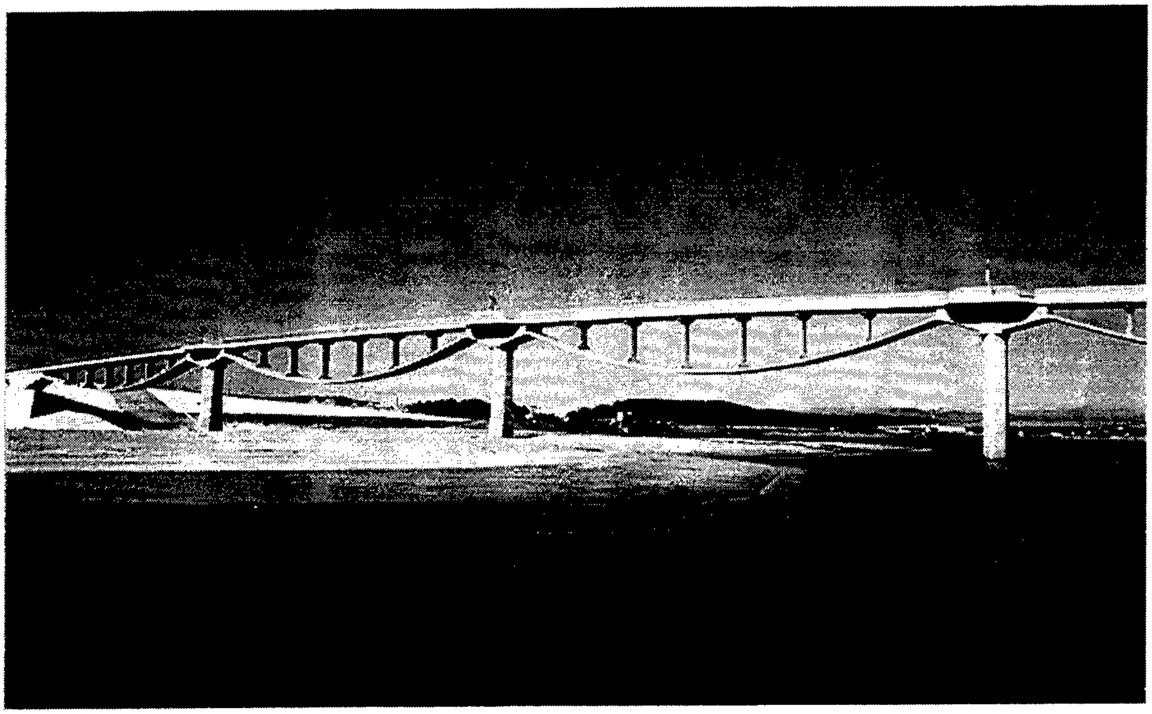
The Natorigawa bridge is a panel stayed prestressed concrete bridge built over Natorigawa river carrying a railway line. Panel stayed bridges are similar to extradosed prestressed concrete bridges (Odawara Highway Bridge) excepting that the external cables are embedded in concrete. Composite action between panels and stay cables help reduce fatigue, reducing the number of external cables that are required. Composite action also increases the stiffness of the structure reducing the deformations due to live load, thereby enhancing the structural performance.

The bridge was constructed over a period of four years in six phases. In phase one, main girders were launched from the main piers and prestressed; false work was removed at the end of first dry season. In phase two, (during the next flood season) cable towers were constructed on the main piers. In phase three, remaining length of the girders were constructed on temporary support structures built on the river bed the following dry season. In phase four, the supports were removed after introducing tension in diagonal cables which were anchored in the main girders and strung over a saddle structure with steel pipe sheaths constructed monolithic with the tower. In phase five, diagonal panels were cast on a false work during the next flood season. In phase six, the false work was removed and all the remaining cables were prestressed and the bridge was opened to traffic.

Reference

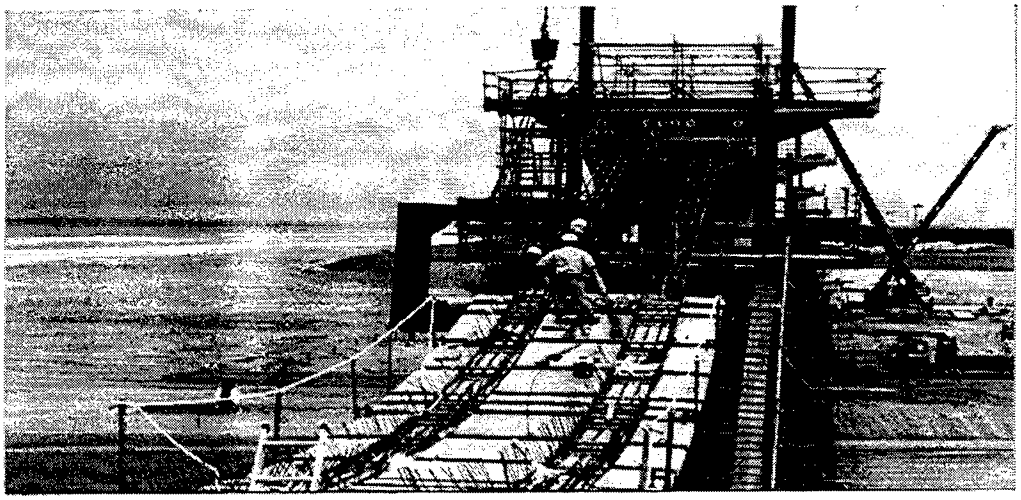
Tadayoshi Ishibashi, Ikuo Iwasaki, Katsunori Takahashi and Tetsuya Kikuchi (1998). "Prestressed Reinforced Concrete Bridge Natorigawa Bridge," *Japan Prestressed Concrete Engineering Association*, National Report, pp. 107-110.

Shiosai Bridge



Location	Shizuoka Prefecture
Type	Pedestrian Stress Ribbon Bridge
Span	61 m (200.13)
Owner	Shizuoka Prefectural Office
Year	1995

Shiosai Bridge



The Shiosai Bridge is a four-span prestressed concrete *stress ribbon* bridge. Stress ribbon bridges are *form resistant* structures (like arches) whose strength and stiffness are achieved from its shape rather than its size. They are primarily economical for longer span recreational bridges in remote areas in the 30m-150m range where falsework cannot be used or requires the use of expensive equipment.

The Shiosai Bridge is essentially an *inverted* suspension bridge with no towers but with the steel cables and hangers replaced by prestressed concrete elements. Because falsework was not used, cables were used instead. The cables were anchored at the abutments and threaded through openings in the pier. Precast stress ribbon segments were then hung from the cables using special hangers and the segments assembled. This served as a working platform on which column segments and roadway slab were erected. The sag to span ratio was kept at 1:10 to minimize horizontal forces at the abutments.

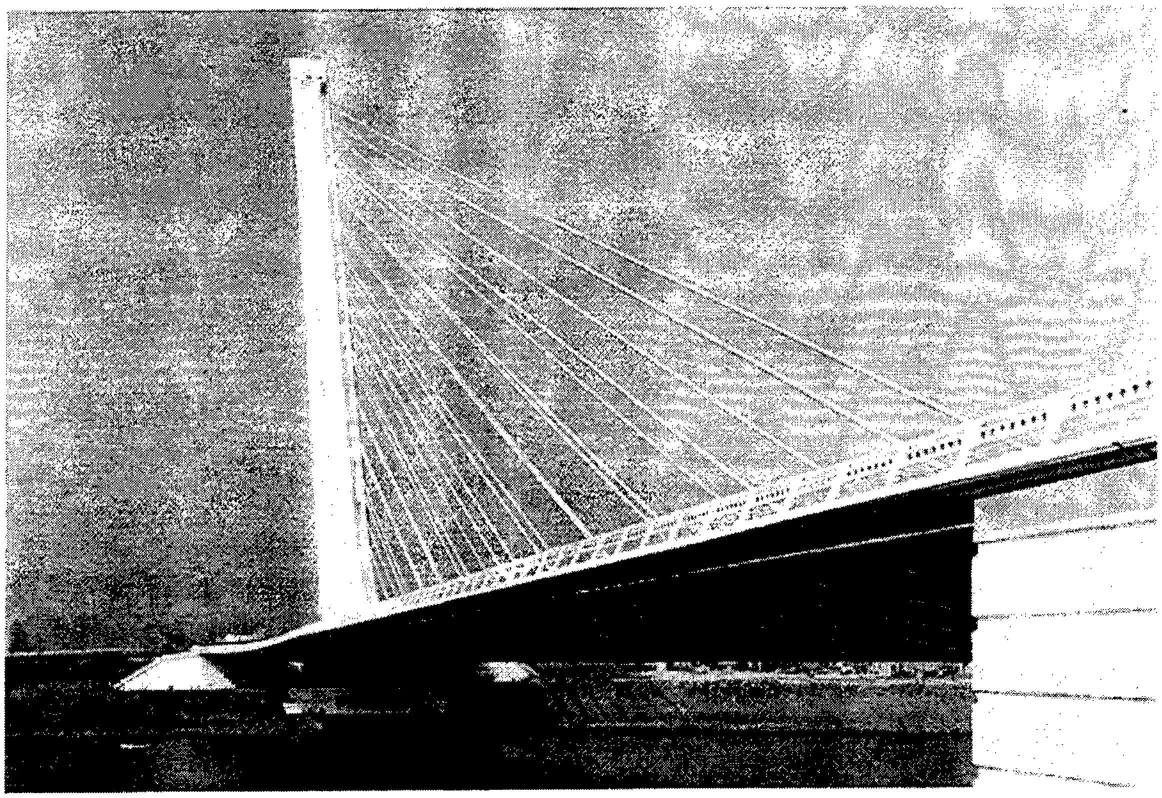
Stress ribbon bridges have also been used for highway bridges. The most notable such structure is the 205 m bridge crossing the Colorado river in Costa Rica. While such structures may be economical, they require complex, multi-stage non-linear analysis.

References

Horiuchi, S, Watanabe, K. and Kondoh, S. (1998). "Four Span Stress Ribbon Bridge With Roadway Slab Decks Shiosai Bridge," *Japan Prestressed Concrete Engineering Association*, National Report, pp. 67-70.

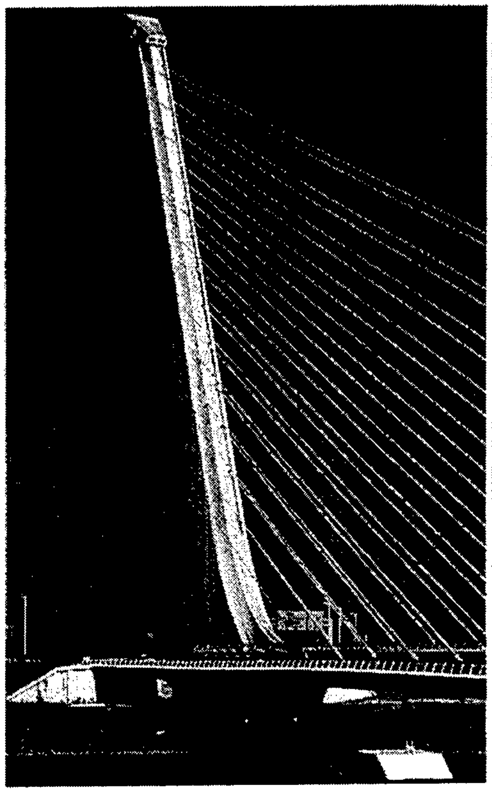
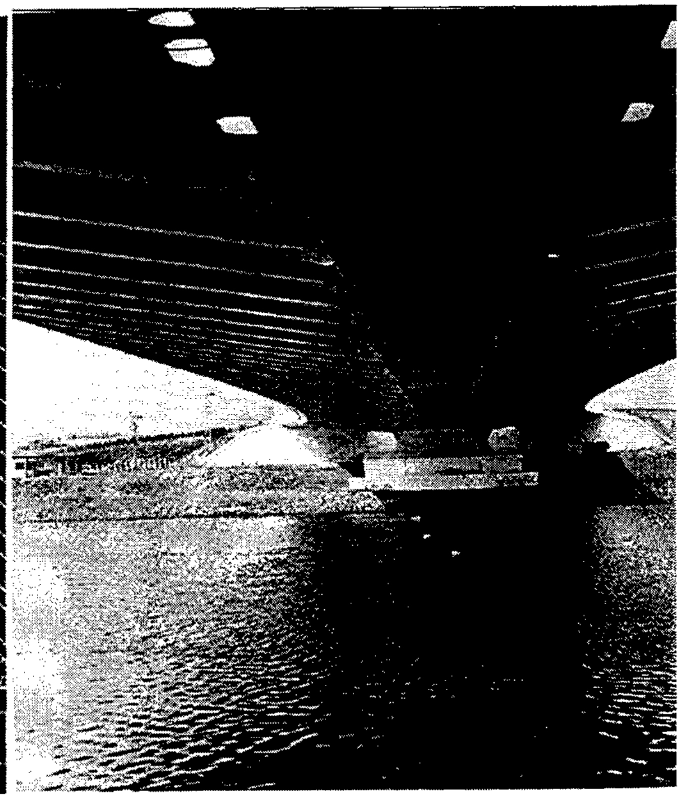
HP Engineering Consultants (1998). Innovation in Practical Engineering, Issue 4, Stress-Ribbon Bridge, Web site : www.easyinternet.net/jpodolak/innov4a.htm.

Alamillo Bridge



Location	Seville, Spain
Type	Cable stayed with inclined tower
Span	200 m (656 ft)
Architect	Santiago Calatrava
Year	1992

Alamillo Bridge

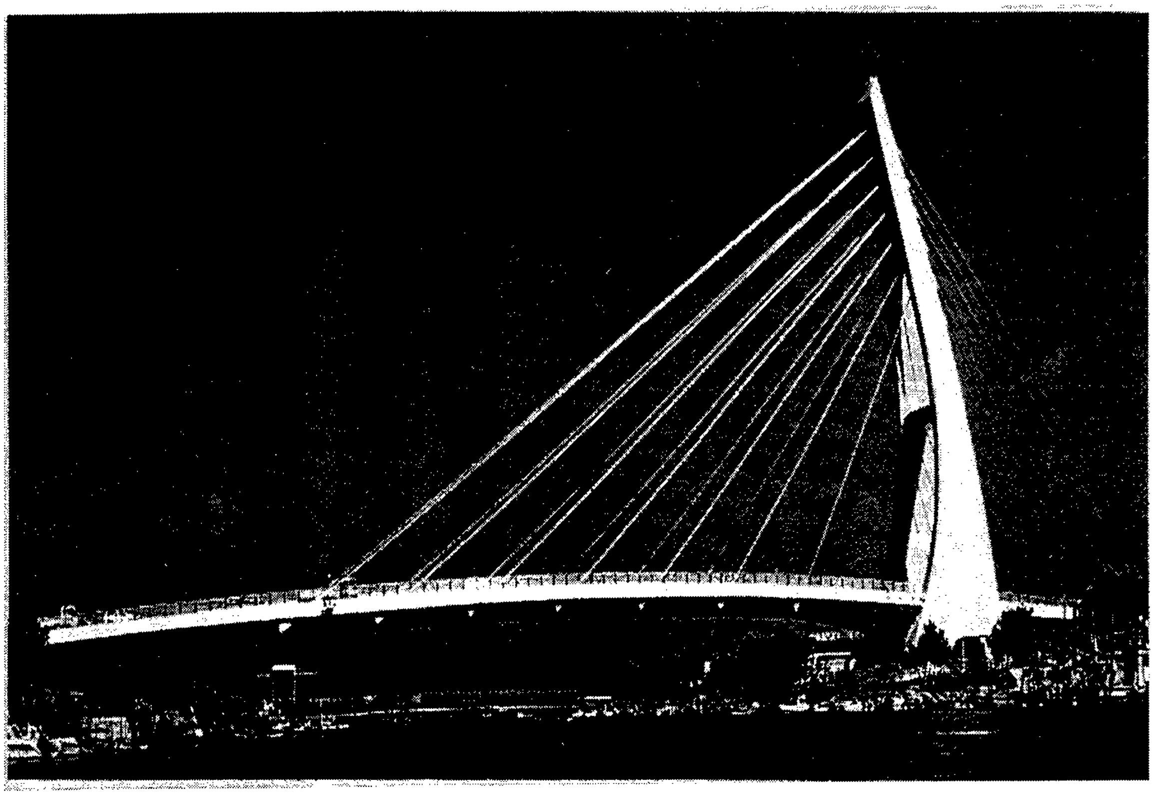
The Alamillo Bridge is a cable stayed structure with no backstays. Instead, the asymmetric cable loads are resisted by the weight of a *backward* leaning tower. The tower's weight pulls its downwards and its backward slope helps to counter the tendency of the stays to bend the tower towards the water. The horizontal component of the cable forces in the deck is balanced by the horizontal component of the tower weight at its base. As a result, the foundation of the Alamillo Bridge only resists vertical loads.

The bridge deck is made of a hexagonal steel box to which the stay cables are attached. The 3.75m wide top side of the hexagonal box beam forms a pathway for pedestrian and cyclists 1.6 m above the carriageway. Steel wings supporting the deck are cantilevered on either side to form the traffic lanes.

Reference

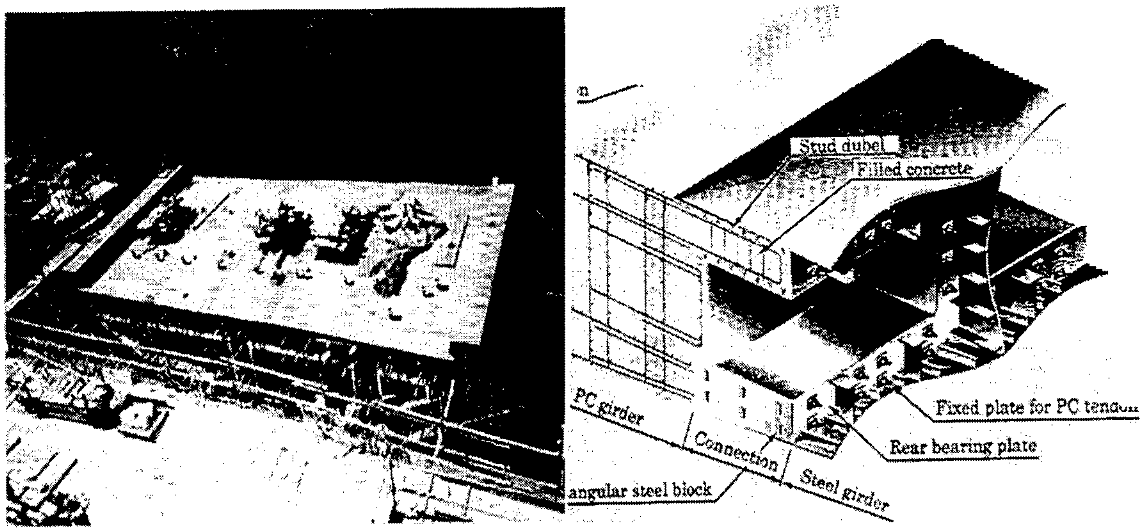
Frampton, K., Webster, A. and Tischhauser, A. (1996). *Calatrava Bridges*, Birkhauser Verlag, Basel, Switzerland, pp. 54-69.

Sun-Marine Bridge



Location	Arai, Japan
Type	Cable Stayed Bridge
Span	144.3 m (473.4 ft)
Owner	Hamanako Motorboat Racing
Year	1996

Sun-Marine Bridge



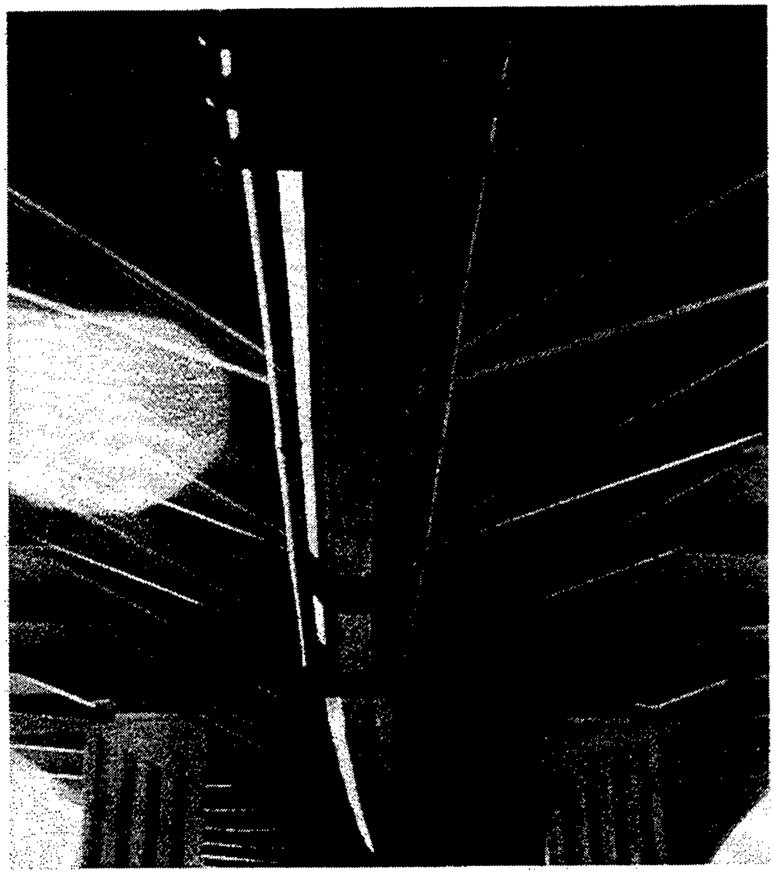
The Sun-Marine bridge is a hybrid, two span continuous, hollow-slab, cable-stayed bridge with a main span of 144.3 m over a river and an access span of 54.3 m over land. Light weight steel box girders with orthotropic steel decks were used for the larger main span whereas prestressed concrete girders were used for the smaller access span to cope with the unbalanced span arrangement.

The connection between the steel box girder and concrete girder was achieved by an end bearing plate welded to the steel box. Galvanized steel stay cables passing through a curved concrete tower support the deck. The curved shape was given to make the tower resemble the sail of an yacht to blend with the image of the region.

Reference

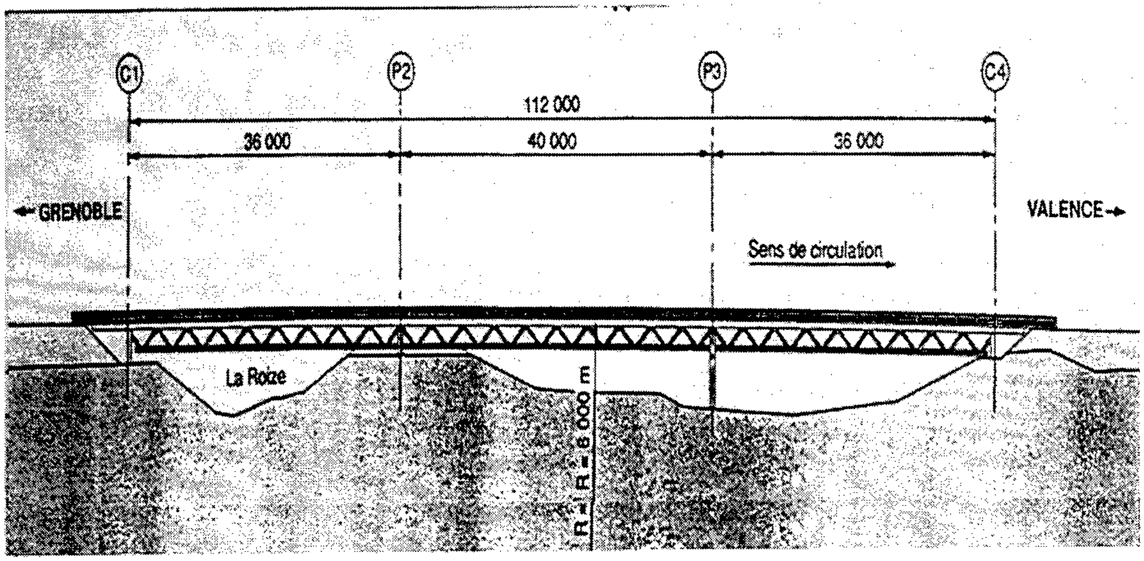
Osamu Miuyano, Shuichi Takefusa and Hideaki Irikura (1998). "Cable Stayed Bridge with a Hybrid Structure -Sun-Marine Bridge-," *Japan Prestressed Concrete Engineering Association*, National Report, pp. 87-90.

Roize Bridge



Location	Roize River, A49, France
Type	Composite Space Truss
Span	40 m (131 ft)
Consultant	Campenon Bernard
Owner	French Highways
Year	1990

Roize Bridge



The Roize Bridge is a new type of composite space truss bridge developed by J. Muller International, Paris that combines concrete, structural steel and prestressing. The bridge is the culmination of a 10 year search by the French Highway Administration to lighten bridge decks in medium span bridges.

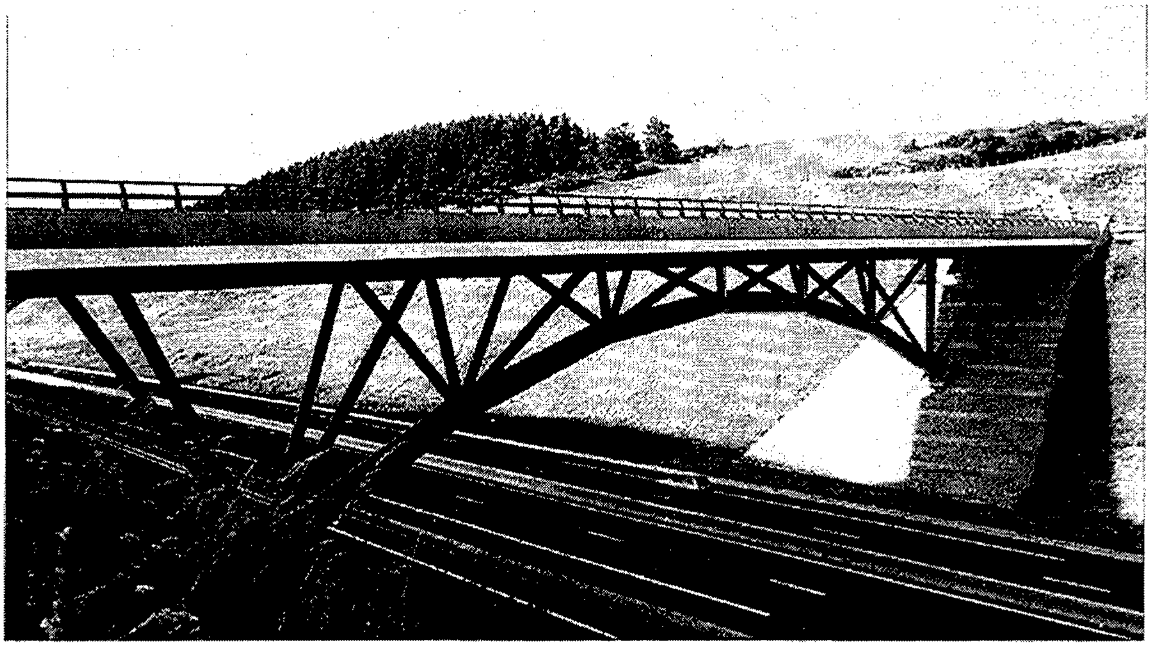
The three span, two-lane, Roize Bridge has a 12.2m (40 ft) wide precast, prestressed concrete deck slab that acts compositely with a steel space truss. The space truss is composed of a single hexagonal lower flange, two inclined Warren-type trusses and transverse floor beams spaced 4 m (13 ft) apart. The bridge is additionally post-tensioned longitudinally by draped tendons continuous over three spans.

The bridge employed a unique modular construction method in which the space truss was prefabricated in 6 ton tetrahedral units that were delivered by truck to the construction site. Each tetrahedral unit comprised a transverse floor beam, four diagonal truss members and a 4 m section of the hexagonal lower flange. Complete details may be found in the reference. It is believed that the bridge is suitable for much wider and longer spans including cable-stayed bridges.

Reference

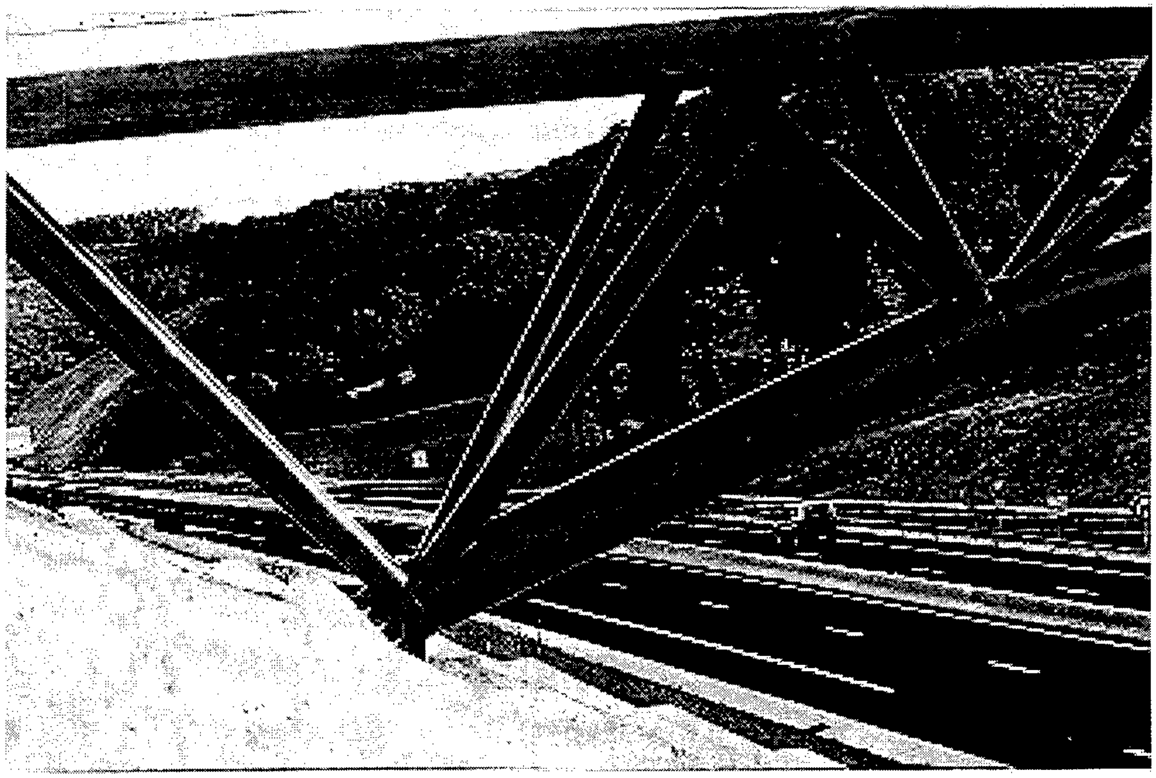
Muller, J.J. (1993). "Bridge to the Future," *ASCE Civil Engineering*, Vol. 63, No 1, pp. 40-42.

Antrenas Bridge



Location	Marvejols, France
Type	Tubular steel arch
Span	56 m (183.7 ft)
Designer	Michel Virlogeux
Year	1996

Antrenas Bridge



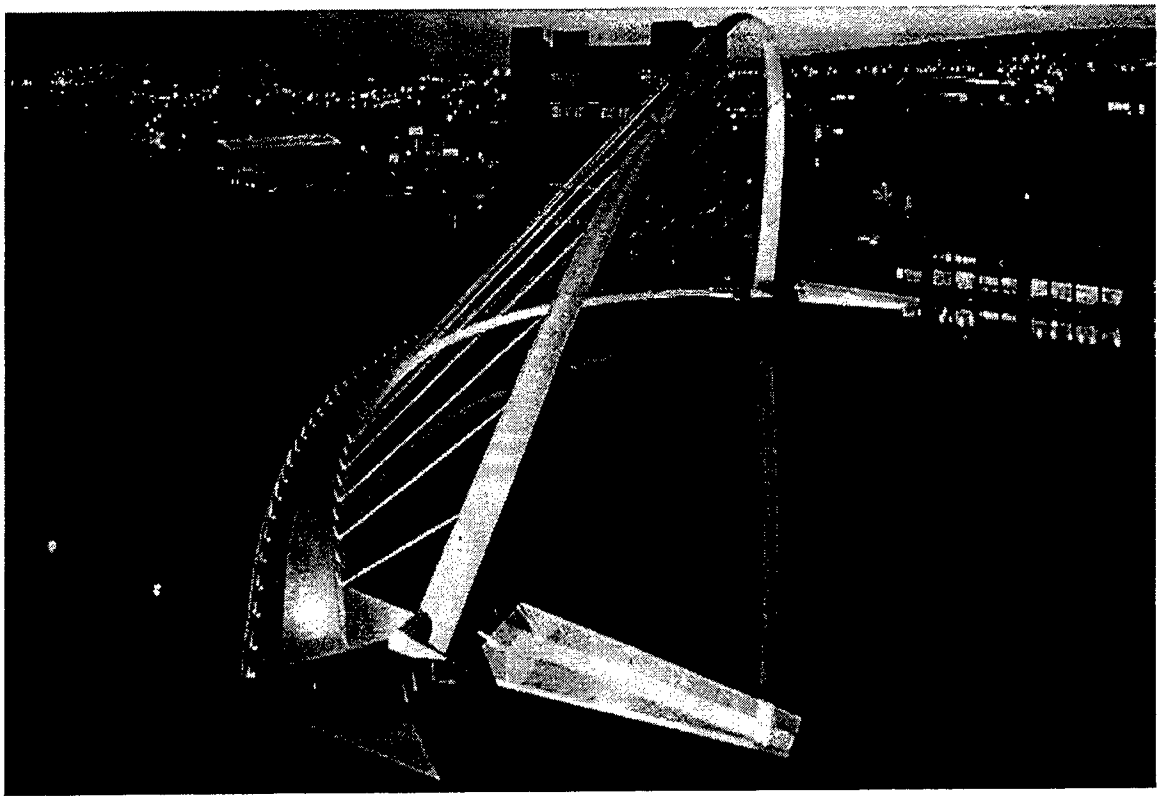
The Antrenas bridge is a tubular steel arch structure supporting a prestressed concrete deck slab. The deck slab is connected to the polygonal arch by tubular truss members. The steel arch section is 1.2 m in diameter and 32 mm thick. Tubular struts 500 mm diameter and 16 mm thick form the spatial steel truss that transfer deck loads to steel arch. The arch sections near the traffic lanes are filled with concrete.

The concrete deck is 11 m wide and 35 cm thick with the thickness at the ribs increased to 95 cm. The deck is longitudinally and transversely prestressed and rests at the ends on cross beams supported by neoprene bearings.

Reference

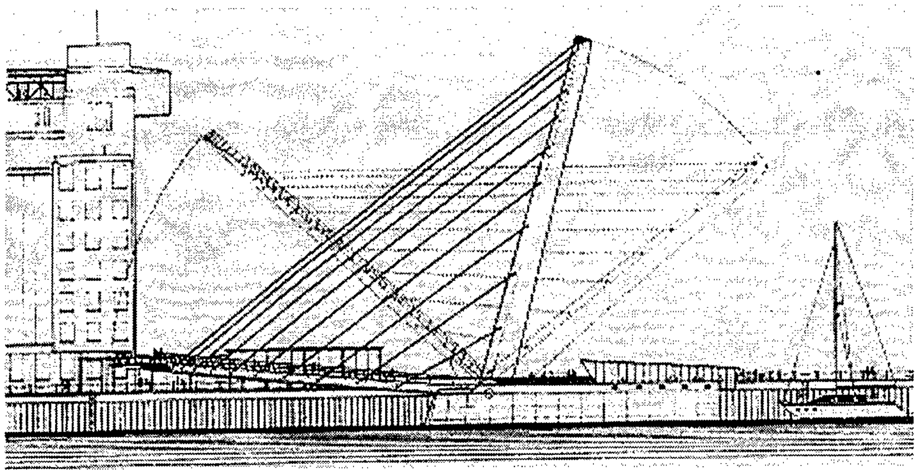
M. Virlogeux, E., Bouchon, J., Berthellemy, J. and Resplendino (1997). "The Antrenas Tubular Arch Bridge," *Structural Engineering International*, Zurich, Switzerland, Vol 7, Number 2, pp.107-109.

The Tyne Millenium Footbridge



Location	Gateshead, England
Type	Pedestrian
Material	Steel arch
Span	100 m (328 ft)
Consultants	Gifford and Partners
Architects	Chris Wilkinson Architects
Year	1998-2000

The Tyne Millenium Footbridge



The Tyne Millennium Foot Bridge is a new type of steel bascule bridge that is to be built over the River Tyne joining Newcastle and Gateshead in northeast England. Detailed design work is still in progress and construction is scheduled to begin in summer (1998) with an expected completion date of September 2000.

The bridge essentially consists of a pair of parabolic arches spanning 100m - one forming the deck and the other supporting it with stay cables. Both of these arches pivot from common springing points located in the pile cap. The 600 ton bridge slowly opens like a giant eyelid about the pivots. In the open position,
the two arches rotate 40° and the suspension cables become horizontal. In its fully open position, the bridge provides a 25m vertical clearance.

The parabolic arch has a kite shaped section tapering both in plan and elevation. It is built with steel members up to 35mm thick. Provisions for maintenance and inspection are incorporated within the arch.

The deck is a box section tapering both in plan and elevation; it also provides a recessed path for cyclists. A new epoxy paint system is under consideration to enhance durability and reduce maintenance of the structural steel work. Additional details may be found in Curran (1998).

References

Curran, P. (1998). " The Millennium Footbridge, " *New Steel Construction*, Vol 6, No.1, Feb/Mar, pp. 24-26.

Staff (1997). " Eye Opener, " *Bridge Design and Engineering,* No.7, May, p. 75.

10.3 New Materials

There have been significant developments in new materials used in bridge construction in the last decade. Where traditional materials such steel or concrete are enhanced, there is immediate acceptance by the profession. Such is the case for high performance steel (Chapter 4) that has already been used in the construction of two bridges, the Snyder South Bridge in Nebraska and the Martin Creek Bridge in Tennessee. Brief descriptions of both these bridges are included in this section.

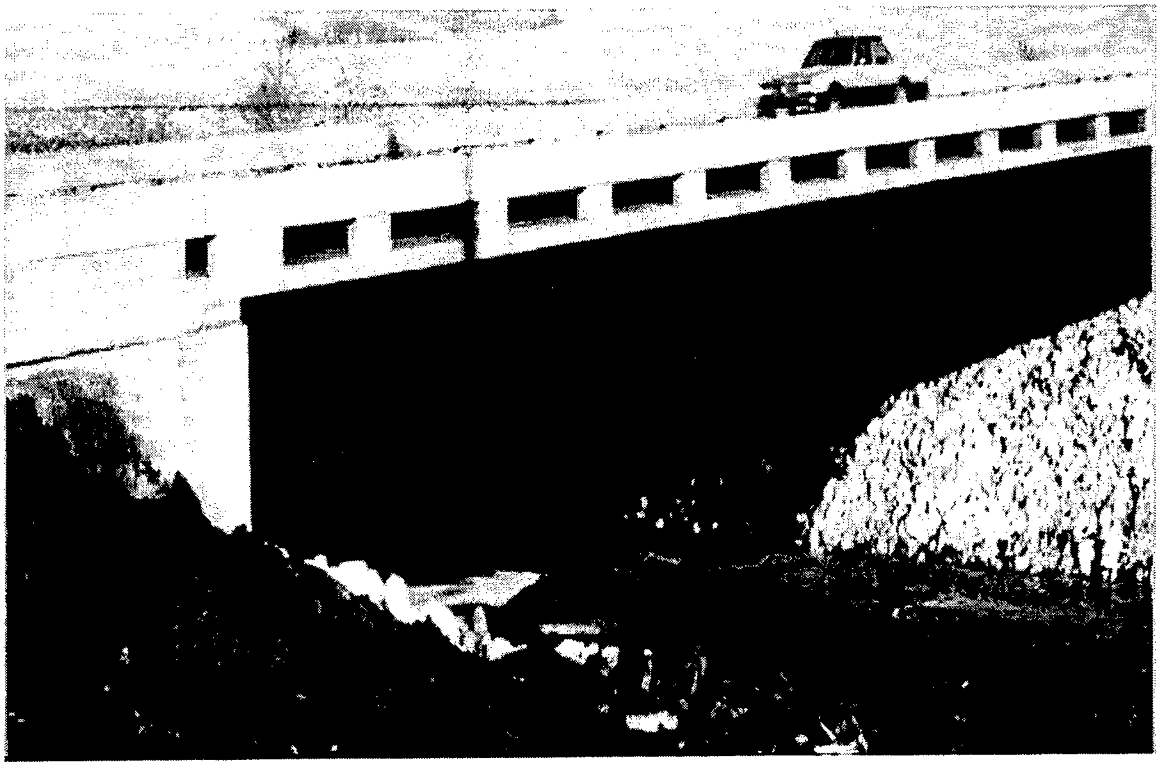
Newer materials are rightly viewed with suspicion and are generally held to a higher standard. A truly new class of material under development that shows great promise are fiber reinforced polymers (FRPs) in which high strength fibers are embedded in a polymer resin. Among fibers successfully used are aramid, carbon and glass. Many demonstration structures utilizing FRPs have been built in Japan, Europe and in North America.

Two different types of FRP bridge superstructures applications are described. The first is the *all FRP deck.* Two examples of its application are included. These are the Wickwire Run Bridge and the Laurel Lick Bridge - both in West Virginia. The second is an application of *FRP as a reinforcing material for concrete.* An example of its use in a highway bridge is the McKinleyville Bridge in West Virginia. As FRPs tend to have moduli considerably lower than that of steel, its use as a reinforcing material is unlikely to be economical in the long run. In prestressing applications the lower modulus leads to lower prestress losses. FRP has been more widely used in prestressing applications and several demonstration bridges have been built. However, it is debatable whether FRPs can compete against conventional materials in the near future.

FRP material has enormous promise as *tension members* in cable-stayed and suspension bridges. An important advancement was recently made in Switzerland where two of the twenty four cables in a cable stayed bridge were made of carbon fiber. A brief description of the Storck Bridge utilizing carbon fiber reinforced polymer cables is included in this section. This structure is being carefully monitored and if it performs as expected, it will promote the use of carbon fiber. Carbon's excellent corrosion resistance, coupled with its high tensile strength, superior fatigue resistance and light weight will also make it possible to span much greater distances.

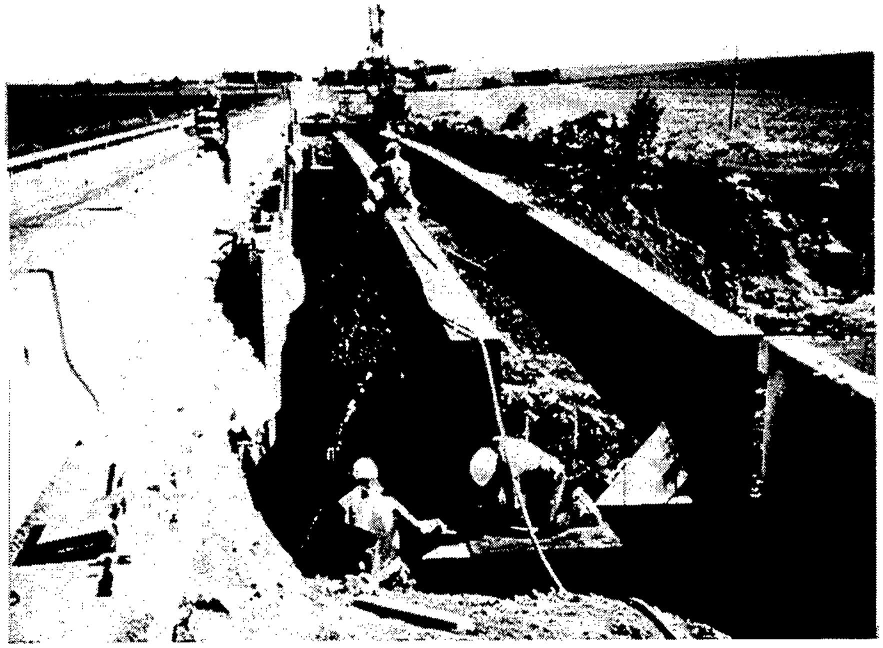
Aluminum is also being given serious consideration as a bridge material, particularly in remote locations in Scandinavian countries where its light weight and corrosion resistance make it cost effective. As aluminum may be three times as costly as steel its use as main structural elements is less likely in this country. In the United States, aluminum is however, being considered for deck replacement (see Chapter 5).

Snyder South Bridge



Location	Route 79, Dodge County, Nebraska
Material	High Performance Steel
Type	Plate Girder
Span	45.7 m (150 ft)
Owner	Nebraska DOR
Engineer	NE DOR, Bridge Div
Year	1997

Snyder South Bridge



Snyder South Bridge is the first bridge to be built in the United States using the newly developed 70 ksi high-performance, weathering steel (HPS) referred to as HPS-70W. Although 70 ksi steel has been available for some time, potential fabrication problems limited its use. The new HPS-70W steel has significantly higher fracture toughness and may require no preheat, a factor that greatly facilitates field welding.

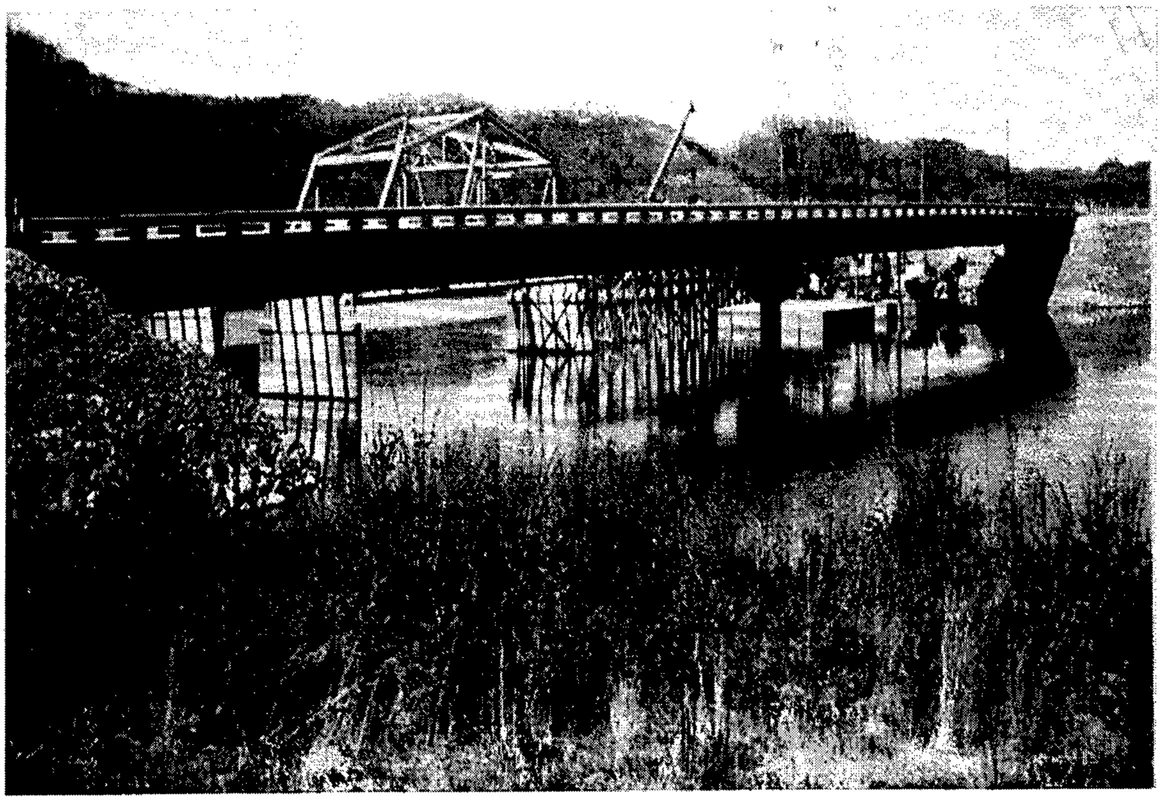
The Synder South Bridge is a simple span plate girder structure that uses five HPS-70W I-girders. Construction of the bridge began in May 1997 and it was opened to traffic on December 15, 1997.

References

Azizinamini, A. (1998). "High Performance Steel:New Horizon in Steel Bridge Construction," TRB News, No. 194, Jan-Feb, p. 24.

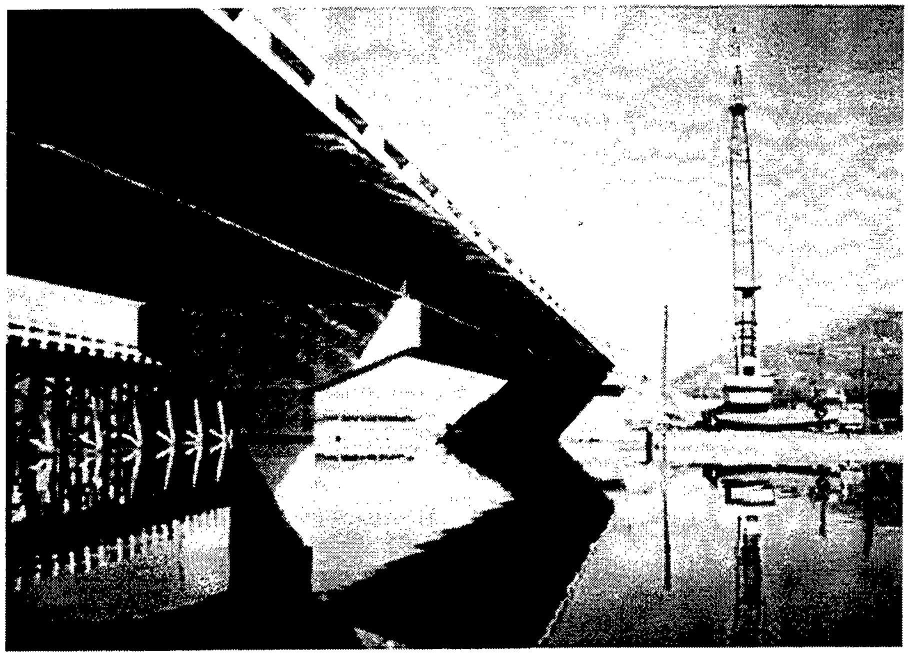
Steel Bridges (1998). "High Performance Steel Score Board," Spring, Steel Bridge Forum, Washington, D.C.

Martin Creek Bridge



Location	Tennessee, Jackson County
Material	High Performance Steel
Type	Plate Girder
Span	71.9 m (236 ft)
Owner	Tennessee DOT
Engineer	TN DOT
Year	1998

Martin Creek Bridge



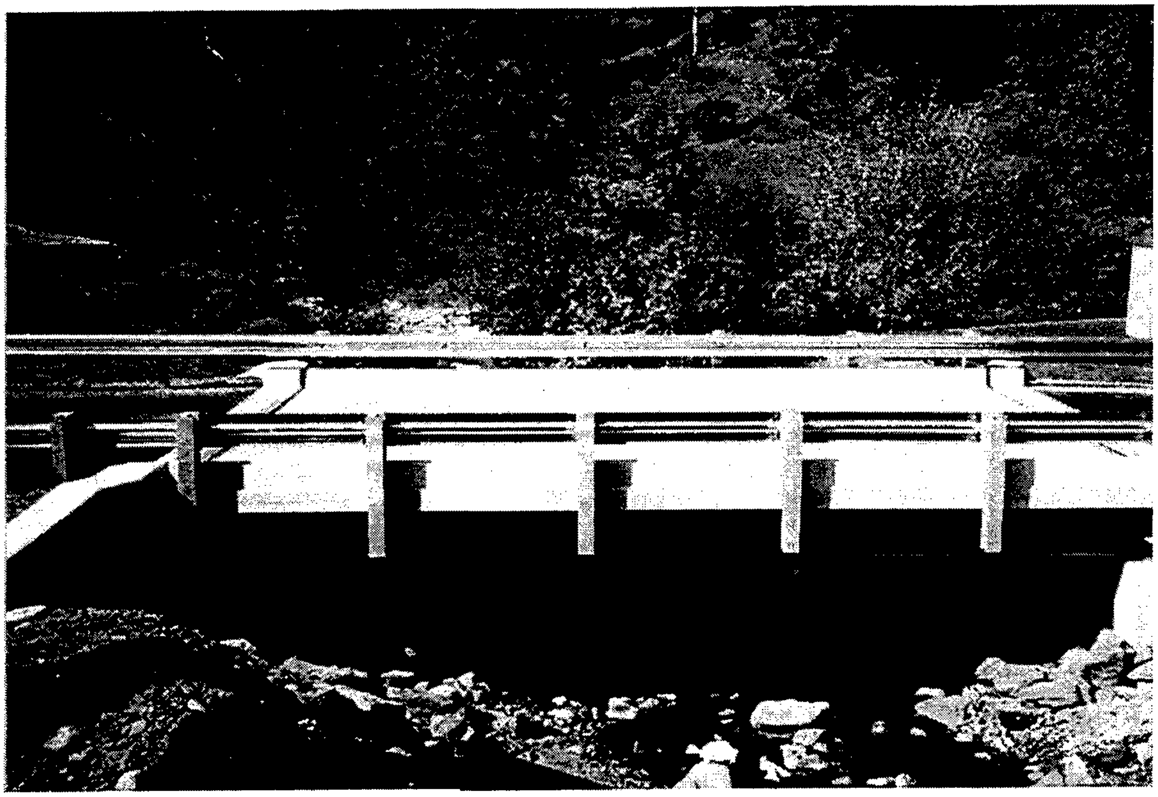
The Martin Creek Bridge is the second HPS steel bridge to be built in the country and the first in Tennessee to use the LRFD specifications. The welded plate girder bridge uses HPS 70W steel that became available for testing in 1996. Cross-frames utilize ordinary 50 ksi steel.

The bridge carries a 30 ft 4 in wide composite deck supported by three plate girders spaced 10 ft 6 in apart. Grade 70 steel required a greater web thickness but because of the higher shear strength, the full bending capacity could be used. Additionally, there were no reductions in bending capacity due to the unbraced length of the compression flange. The structure was originally designed using 50 ksi weathering steel; the new design using 70 ksi steel provided 24.2% savings in weight, a 31.4% reduction in lifting weight and 10.6% savings in overall cost. In common with other TDOT structures, the bridge is jointless having integral, pile-supported abutments. The bridge was opened to traffic on February 9, 1998.

Reference

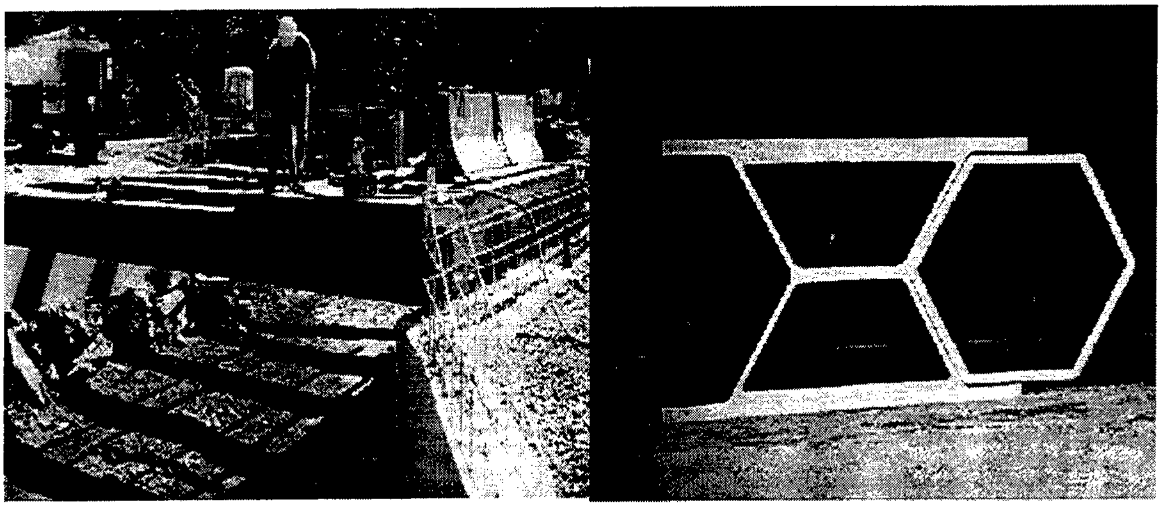
ED Wasserman (1997). "Tennessee Experience With High Performance Steel Bridge," *New Horizons in Steel Bridge Construction*, Regional Seminar, Mid-America Transportation Center, Lincoln, Nebraska, Oct 6-7.

Wickwire Run Bridge



Location	Taylor County, WV
Type	FRP Deck
Span	9.1 m (29.8 ft)
Owner	WVDOT
Year	1997

Wickwire Run Bridge



The Wickwire Run Bridge is one of two demonstration bridges built in West Virginia to assess the feasibility of using fiber reinforced polymer (FRP) decks. In this structure, the E-glass/vinylester FRP modular deck is supported by four longitudinal steel stringers spaced 1.83 m (6 ft) apart. The modules were each 2.44 m (8 ft) wide and their length matched the 6.6 m (21.6 ft) width of the two-lane bridge.

The 200 mm (8 in.) thick composite deck cross-section is made of full-depth hexagons and half-depth trapezoids. This provides a load carrying capacity that is several times that of conventional reinforced concrete yet it weighs only 1.05 kN/m^2 (22 psf) - less than a quarter of the weight of an 200 mm (8 in.) reinforced concrete slab.

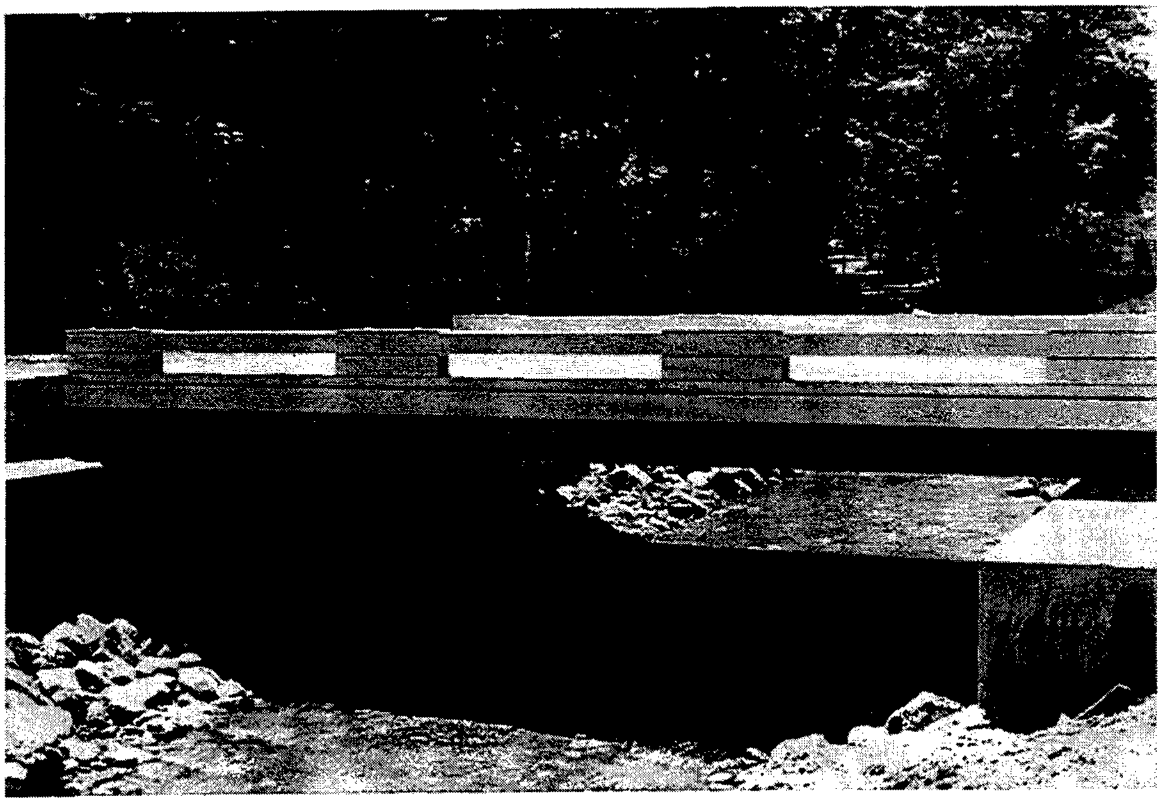
The FRP modules were placed side by side and bolted to each other. They were attached to the steel stringers by specially engineered 13 mm fasteners and polyurethane adhesive. Prior to bonding, the contact surfaces between the steel stringers and the FRP modules were sandblasted for better adhesion. The performance of the FRP deck and its connections is being carefully monitored.

References

Roberto Lopez-Andio (1997). "Wickwire Run Bridge," *Press Release*, CEMR, West Virginia University, Nov 6.

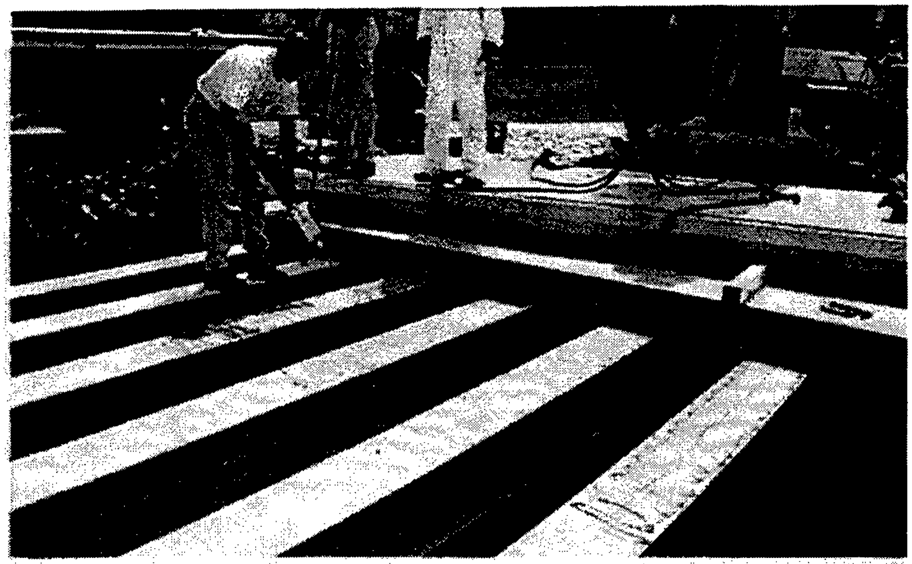
Roberto Lopez-Andio and Hota V.S. Ganga Rao (1997). "Design and Construction of Composite Material Bridges," in *Recent Advances in Bridge Engineering* (Eds. U. Meier and R. Betti), Columbia University Press, July, pp. 269-276.

Laurel Lick Bridge



Location	Lewis County, WV
Type	All FRP Deck
Span	6.1 m (20.0 ft)
Owner	WVDOT
Year	1997

Laurel Lick Bridge



The Laurel Lick Bridge is the first of two short-span modular FRP bridges constructed in West Virginia. The composite modular deck is supported on wide flange FRP stringer I beams. This bridge is 6.1 m (20 ft) long and 4.9 m (16 ft) wide. Details of the modular deck are similar to those in the Wickwire Run Bridge. As before, polyurethane adhesive was used to bond the FRP modules to the stringers. The material cost of the modular deck is higher than that of conventional reinforced concrete deck but its high strength to weight ratio reduces construction costs. Maintenance and replacement costs are also lowered due to FRP's excellent resistance to corrosion and good fatigue strength.

Prefabricated light weight modular construction brings about significant savings by minimizing traffic disruptions and commerce. Repair and construction of modular bridges can be achieved rapidly unlike concrete deck bridges. The composite deck in Laurel Lick Bridge was installed in *just 5½ hours*.

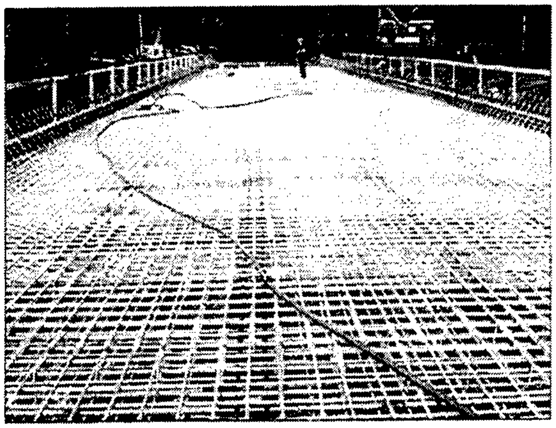
Reference

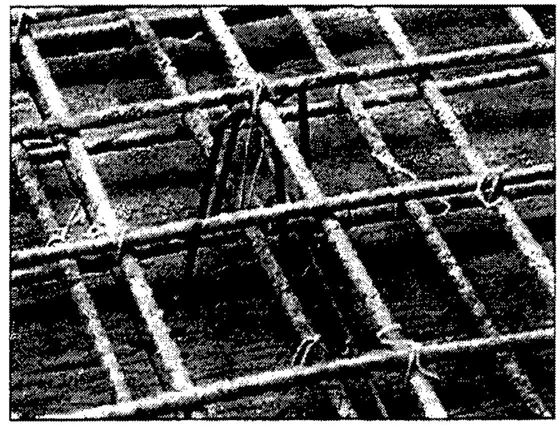
Roberto Lopez-Andio (1997). "Laurel Lick Bridge," *Press Release*, CEMR, West Virginia University, June 6.

McKinleyville Bridge

Location	Brooke County, WV
Type	FRP Reinforced Concrete Deck
Span	22.3 m (73.2 ft)
Owner	WVDOT
Year	1996

McKinleyville Bridge





The McKinleyville Bridge is the first vehicular bridge in the US to use a FRP reinforced concrete deck. The deck is placed on Grade 50 steel-rolled beams spaced at 1.5 m (5 ft) center to center. The design required a deck thickness of 231 mm (9 in.) and No. 4 (13 mm) diameter bars at 150 mm (6 in.) spacing as the main reinforcement. Distribution reinforcement consisted of 10 mm (0.4 in.) straight bars and long loops to hold top and bottom reinforcement, leaving a clear cover of 38 mm (1.5 in.) and 25 mm (1 in.) at top and bottom respectively. The design is based on the limitations proposed by CFC (Construction Facilities Center of West Virginia) after extensive laboratory testing.

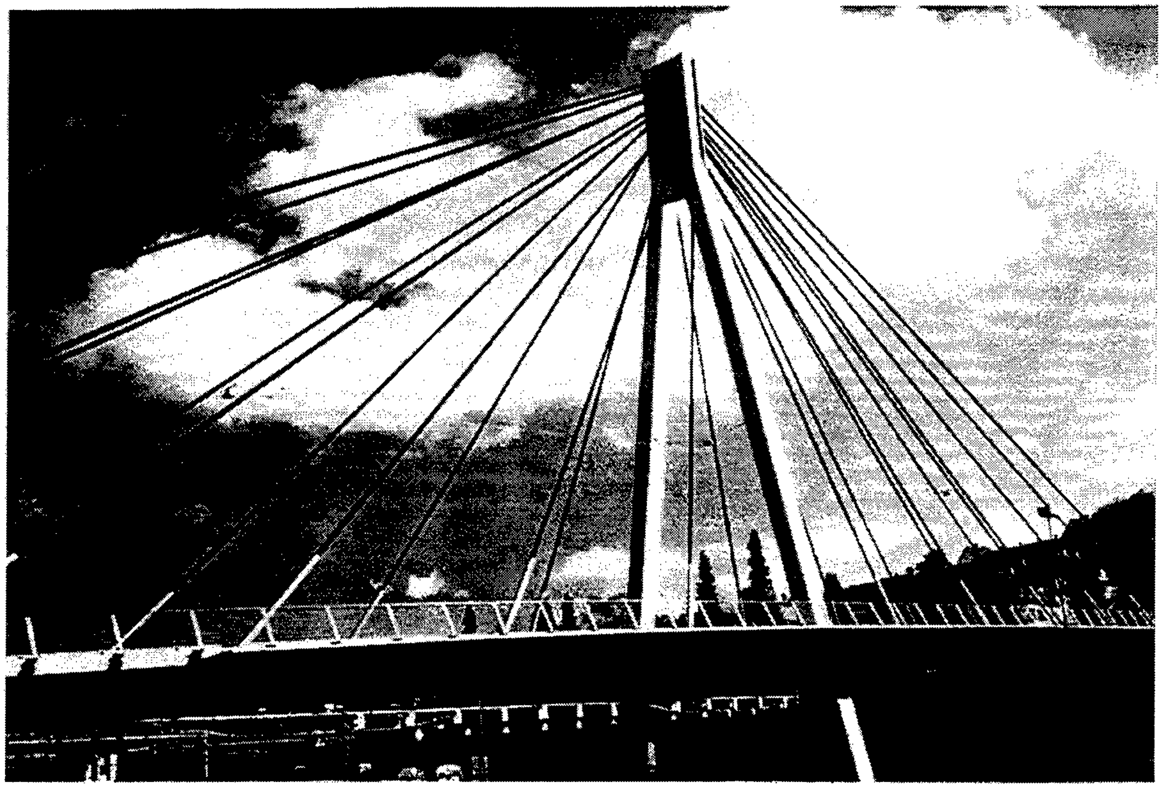
Two types of FRP bars, C-bars and sand-coated bars, were used. C-bars are made of E-glass/polyester resin and sand coated bars are made of E-glass/unsaturated polyester resin. Both types of reinforcing bars have a tensile strength of 560 MPa (80 ksi), a tensile modulus of 42 GPa (6090 ksi) and a bond strength of 10 MPa (1450 psi). Concrete strength of 31 MPa (4500 psi) was used for the deck.

Epoxy coated chairs were placed every 13.1 m (4 ft) to avoid excessive deflections due to construction loads and prevented the light weight FRP mesh from floating when the concrete was vibrated. Precautionary measures were adopted for resisting crack growth and crack formation. Load tests were conducted for monitoring the bridge. In addition, it was inspected for signs of environmental deterioration. Load tests revealed that the bridge performance was as desired. However long term behavior is to be studied. Additional details may be found in the reference.

Reference

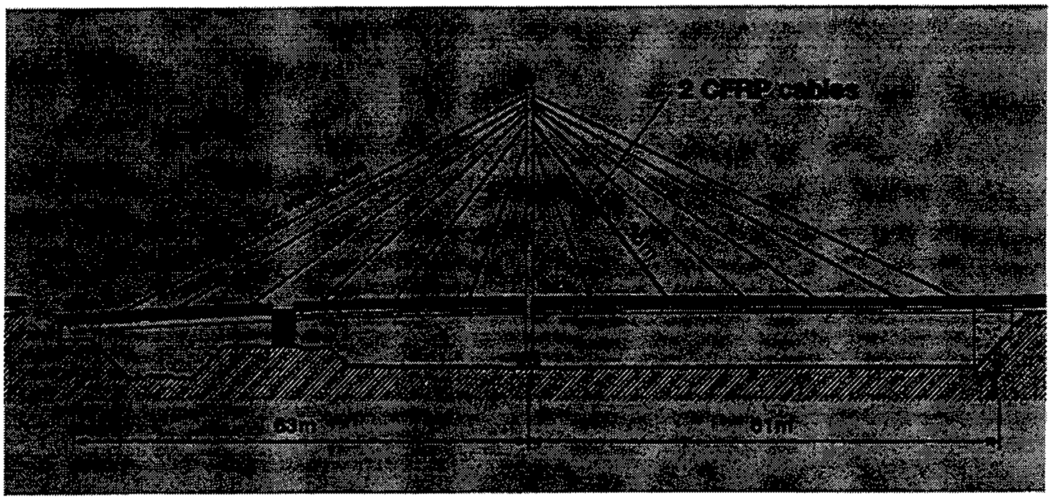
GangaRao, H. V. S., Thippeswamy, H. K. and Jason, M. S. (1998). "FRP Reinforcement in Bridge Deck," *Concrete International*, Vol 20, Number 6, June, pp. 47-50.

Storck Bridge



Location	Winterthur, Switzerland
Type	Cable stayed
Material	Carbon and steel cables
Span	63 m (206.7 ft)
Consultants	EMPA Bureau BBR Ltd., Zurich Höltschi & Schurter AG, Zurich
Year	1996

Storck Bridge



This is the first bridge to use carbon fiber cables. Two of the twenty four cables are made of carbon; the remaining twenty two, are conventional steel. Carbon fiber is a lightweight, high strength, high modulus, high corrosion resistant material with outstanding fatigue characteristics. It is also anisotropic, somewhat brittle and has a lower coefficient of thermal expansion than steel.

The low transverse strength of the carbon fiber posed particular challenges in the development of a suitable anchorage system that allowed loads to be safely transferred to the cables. A patented system was developed that creates a uniform shear stress distribution along the full length of the anchorage. The cable cross-section consists of 241 parallel carbon fiber reinforced plastic (CFRP) wires each having a diameter of 5 mm. The nominal breaking load of a cable is 13600 kN, its modulus, 168 GPa. The bridge is being carefully monitored to assess the performance of the CFRP cables. No significant change in performance was observed after one year of service.

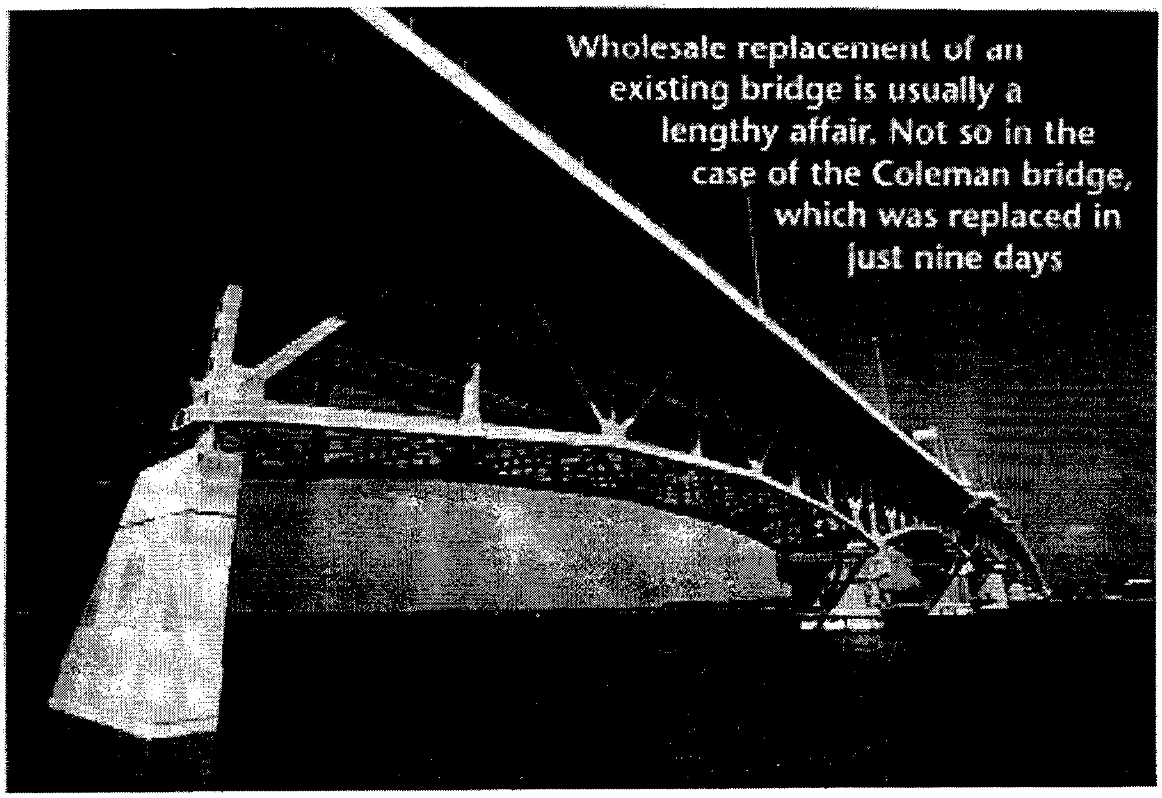
Reference

Sennhauser, U., Anderegg, P., Brönimann and Nellen, P.M. (1998). "Monitoring of Storck Bridge with Fiber Optic and other Electrical Resistance Sensors," in *Recent Advances in Bridge Engineering (Ed U. Meier and R. Betti)*, Columbia University, New York, NY, pp. 368-375.

10.4 Construction

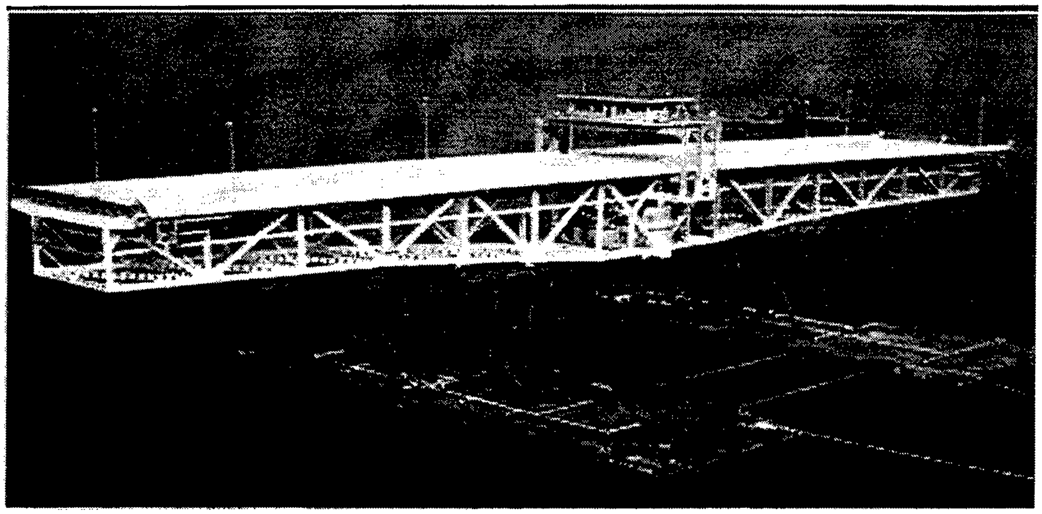
Two examples of innovative design and construction are included. The first is the replacement of the Coleman Bridge where the use of innovative technologies led to successful completion of the project in record time. The second, is the Moore Haven Bridge where the original steel design was replaced by a spliced girder concrete section with a record 98 m span.

Coleman Bridge



Location	Yorktown, Virginia
Type	Deck Truss Bridge
Span	171 m (561 ft)
Consultant	Parsons Brinckerhoff
Owner	VDOT
Year	1996

Coleman Bridge



The George P. Coleman bridge was originally constructed in 1952. This two-lane bridge had become a bottleneck in a four-lane highway system and needed to be urgently replaced. As traffic would be subjected to 75 mile detours during the period of bridge closure, rapid replacement was a *necessity*. Thanks to the use of innovative technologies and materials, the removal of the old section and the installation of the new section was achieved in the remarkably short time of *nine days* with minimal impact on the historic and natural features of the site.

The essential features of the replacement scheme were (1) re-use of existing caissons and (2) float-in construction technique. The re-use of existing caissons led to savings of $20 million as it eliminated the need for a temporary bridge. Float-in construction permitted the replacement of one of the world's largest double-swing span bridges in less than half the time allowed by the contract. The new bridge, complete with roadway and lighting masts, was constructed off-site and floated in place on barges after the old section had been floated out and its pivot bearing removed and replaced.

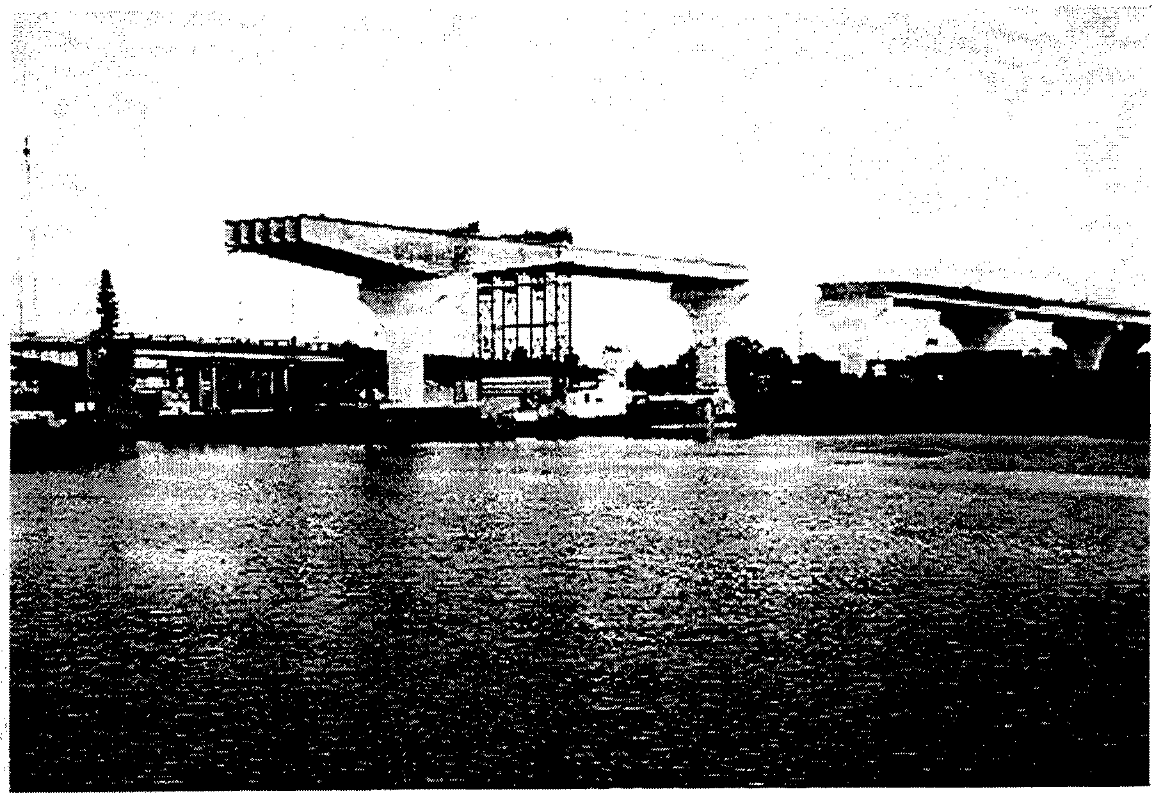
This project has been the recipient of both national and international awards.

References

Michael Abrahams (1997). "Rapid Replacement," *Bridge Design & Engineering*, U.K., Nov, Number 9, pp. 41-42.

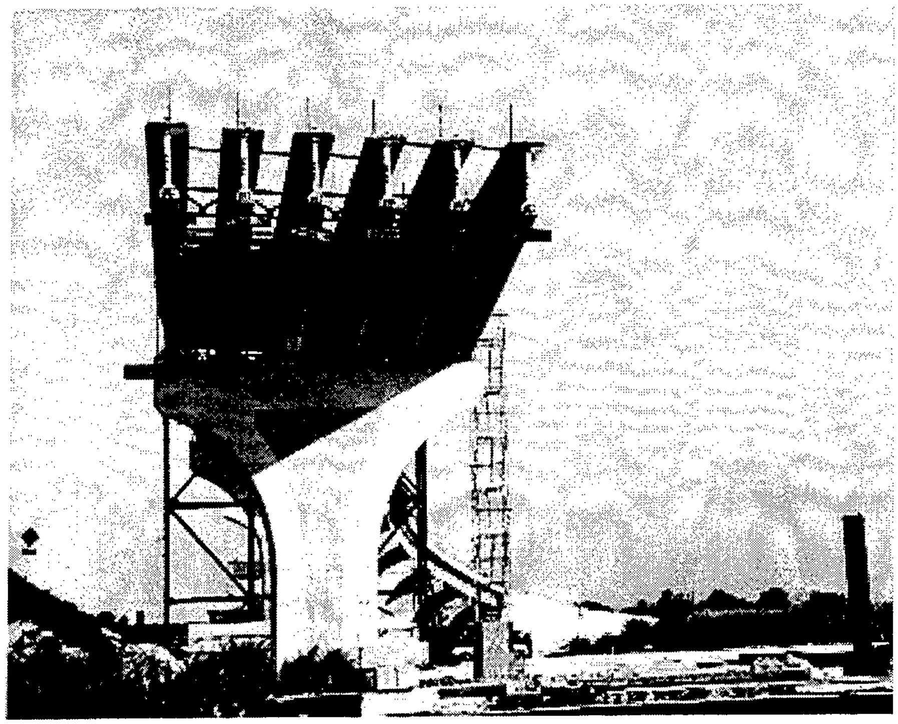
Vincent J. Roney (1997). "Extensive Pre-Assembly Radically Cuts Time," Modern Steel Construction, Chicago, Vol 37, Number 2, pp. 28-34.

Moore Haven Bridge



Location	US 27, Glades County, FL
Type	Post Tensioned Splice Girder
Span	98 m (321.5 ft)
Owner	FDOT
Consultant	Janssen & Spaans
Year	Under construction

Moore Haven Bridge



Spliced girder bridges are used where the girders are too long or too heavy to be transported. Such bridges are not new - the first such bridges were constructed in the early fifties. The Moore Haven Bridge is the longest splice girder bridge to be built in Florida. The previous record was held by the Highland View Bridge built in 1993. This had a channel span of 76.2 m (250 ft).

The segmental bulb T girder used has a maximum depth of 4.57 m (15 ft) at the pier. This gives a span-to-depth ratio 21.3 that is greater than that required for extradosed girders (see *Odawara Blueway Bridge*). Six girders are spaced at 2.7 m (8 ft 10 in) to provide an overall width of 16 m (52 ft 5½ in.). This spacing is somewhat smaller than the 2.9 m (9 ft 6 in) spacing used for the Highland View Bridge. The spliced girder solution proposed by Janssen & Spaans was more economical than the plate girder solution originally proposed.

References

Abdel-Karim, A. and Tadros, M. (1995). "State-of-the-Art of Precast/Prestressed Concrete Spliced I-Girder Bridges," PCI, Chicago, IL, Second Printing.

Garcia, A.M. (1993). "Florida's Long Span Bridges: New Forms, New Horizons," PCI Journal, Vol. 38, No 4, July-August, pp. 34-49.

11. INNOVATIVE SUPERSTRUCTURE CONCEPTS FOR HPS

11.1 Introduction

Two recently completed projects, one funded by the Federal Highway Administration (FHWA) and the other by the American Institute of Steel Construction (AISI), proposed several innovative superstructure concepts for optimizing the use of high performance steel (HPS).

Brief summaries of all proposed concepts are presented in this chapter. Additional information may be found in Refs. 11.1-11.2. Section 11.2 focuses on the FHWA concepts and Section 11.3 on those from AISI [from p. 11.9]. These concepts are referred to in the next chapter that describes the process used in this study to select a concept that was deemed most appropriate for Florida.

11.2 FHWA Concepts

The FHWA sponsored study proposed 16 concepts, not all of which were intended for long span bridges. The concepts cover new ways of utilizing standard sections, prestressing steel girders, replacing webs by corrugated sheeting, the use of tubular elements and cold-formed metals. Table 11.1 provides brief descriptions of the concepts that are numbered from 1 to 16 to allow easy reference. An additional concept proposed for connections is listed as item 17 in this table.

The FHWA concepts were circulated to several engineers in the Tampa Bay area and elsewhere for their comments as to their suitability for this study. The consensus of the comments was best summarized by Mr. Nelson Canjura of T Y Lin International/DRC who wrote:

> *"Overall I was disappointed with the concepts presented. From a practicing engineers' point of view, the concepts were not very practical and lacked details as to their applicability over traditional steel plate and box girders. Some concepts I don't consider "innovative" but variations of past and present practice"* [11.3].

Of the 16 concepts, three received lukewarm support from the panel of engineers who reviewed the concepts. For the lower span end, *the post-tensioned steel girder concept* [Concept #9 on p. 11.5] was selected. For the upper end, the *two tubular truss* concepts [Concepts #15, 16 on p. 11.7-11.8] were recommended.

Table 11.1 Innovative Concepts from FHWA Study [11.1].

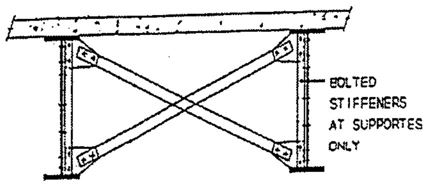
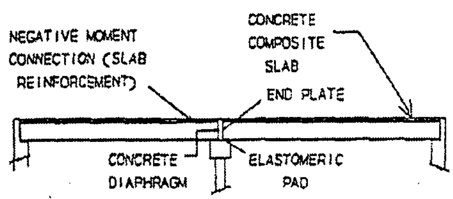
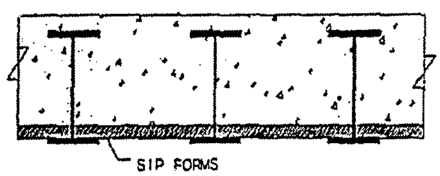
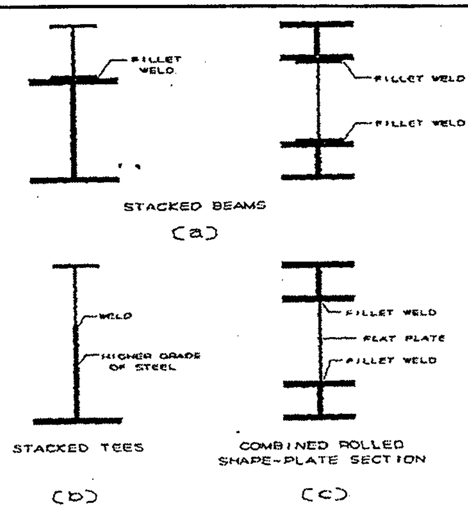
Concept	Description
1  Composite I-girders with all bolted connections  Simple span steel beam continuous for live loads.  Encased beams	These three concepts utilize common materials, familiar fabrication techniques and non-specialist labor. The first uses all bolted connections, the second, eliminates field splices and the third, eliminates bracing. However as fatigue is not a great concern for the type of connection shown, higher fracture toughness may not be as beneficial. While simplicity of the system may reduce labor and fabrication costs, it will increase material and possible construction time.
2  Stacked beam section	This is not a new concept: stacking allows better use of materials. Out of plane stiffness of intermediate flanges may allow use of partial depth cross-frames. Economies result because rolled sections are cheaper and are unlikely to require transverse stiffeners. Weldability and higher fracture toughness of HPS will be harnessed. However, the bridge will be heavier and will increase substructure costs though fabrication costs may be lower.

Table 11.1 (Contd). Innovative Concepts from FHWA Study [11.1].

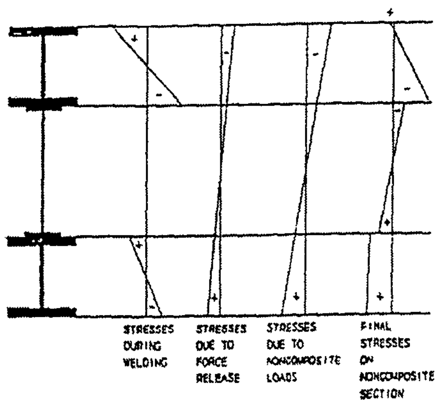
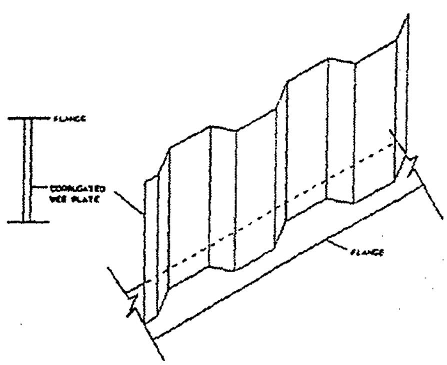
Concept	Description
3  Stress distribution in the indirectly prestressed stacked beams.	This is a variation of the stacked concept in which prestressing is introduced by varying the camber of the different rolled sections and subsequently applying an external load to force the same profile. Prestressing can lead to smaller sections. Other benefits are similar to Concept #2. Fabrication and handling costs are likely to be greater.
4  I-girder with corrugated web.	Thin webs have very low out-of-plane stiffness and can buckle prior to yielding. As a result, the webs have to be strengthened by welding vertical stiffeners greatly increase fabrication costs and reduce fatigue life. The high out-of-plane stiffness of corrugated webs helps eliminate these problems and also allows thinner webs to be used. Additionally, the top flanges are reduced. This results in a lighter structure that is potentially more economical because of reduced fabrication costs.

Table 11.1 (Contd). Innovative Concepts from FHWA Study [11.1].

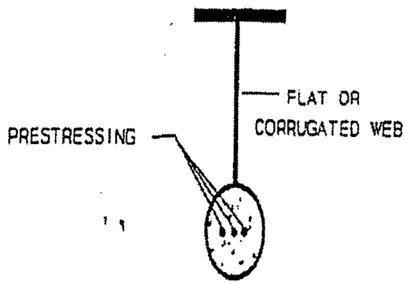
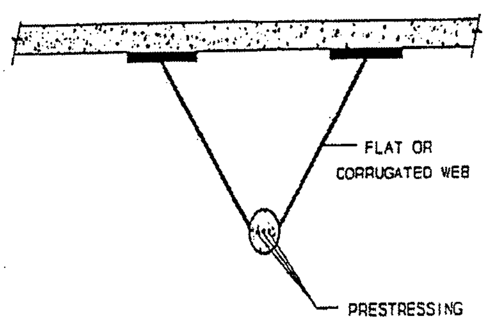
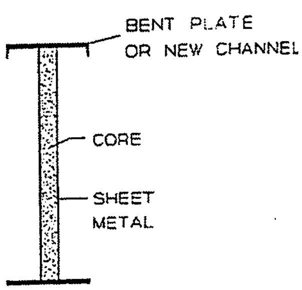
Concept	Description
5  Girder with concrete-filled tubular flange	This concept utilizes commonly available materials to provide a more efficient, lighter section. Prestressing increases internal redundancy and eliminates fatigue and fracture concerns. However, it can introduce compression in the web that can increase buckling susceptibility. Circular flanges prevent accumulation of debris on their top surface and resulting corrosion problems.
6  Triangular box-girder with concrete-filled tubular bottom flange	This is a extension of Concepts 4 and 5 for box sections. Its benefits are similar to those for I sections excepting that the box section will have a much higher torsional stiffness and better load distribution characteristics.
7  I-girder with double sheet-metal web	This sandwich concept has the potential to reduce the web thickness and therefore lead to lighter structures.

Table 11.1 (Contd). Innovative Concepts from FHWA Study [11.1].

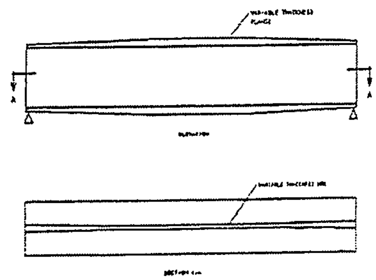
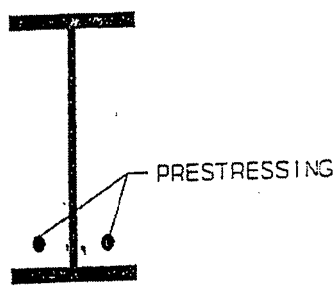
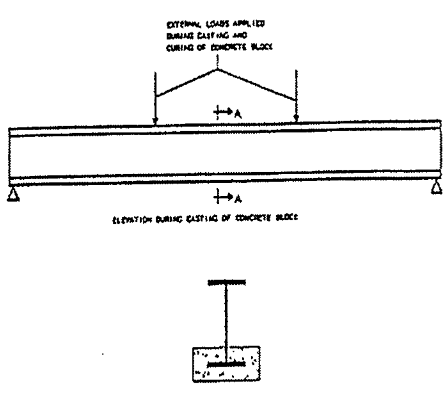
Concepts	Description
8  I-Girder with variable web and flange thickness	This concept utilizes rolled sections of different thicknesses. However, a variable stiffness will make it difficult to analyze and predict behavior. Connections to transverse members is more complicated.
9  Prestressed steel I-girder	Prestressing steel is not a new concept and can lead to lighter sections. Introduction of prestressing may make the web more susceptible to buckling. Corrugated webs may be more suitable. Cost of prestressing hardware may offset weight savings.
10  Indirect prestressing using the inverset system	This concept describes a proprietary inverset system that can be used to introduce pre-stress in simple spans. A concrete slab is cast to encase the tension flange of a steel beam under temporary load.

Table 11.1 (Contd). Innovative Concepts from FHWA Study [11.1].

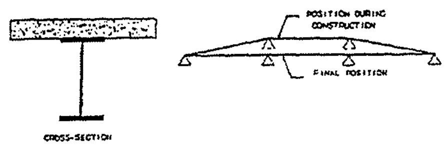
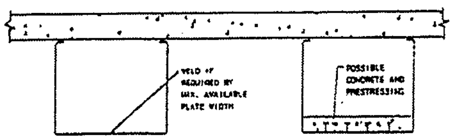
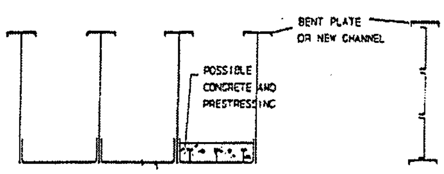
Concepts	Description
11  Indirect prestressing by jacking supports	Indirect prestressing is applied by jacking interior supports prior to casting of the slab. The supports are lowered to their permanent position after the slab has hardened. The resulting system has a larger stiffness and lower stress range in the negative moment region.
12  Standardized multiple tub box girder	Tub box girders are fabricated by cold-forming instead of by welding. Further economy may be achieved by prestressing and by making it composite top or bottom with normal/lightweight concrete. Currently, suitably large cold forming machines are unavailable - the maximum size is 6 m (20 ft) for Grade 485 MPa (70 ksi) and 12 m (40 ft) for Grade 350.
13  Steel girders made of standardized basic components	Utilize mass produced flat plates, U-shaped cold-formed units or new rolled channel shapes to produce girders of different depth and cross-sections. Making the sections composite would further improve efficiency.

Table 11.1 (Contd). Innovative Concepts from FHWA Study [11.1].

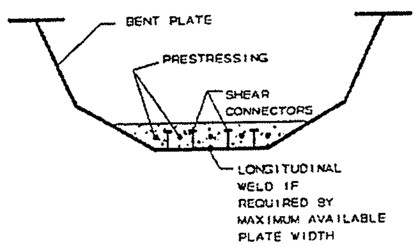
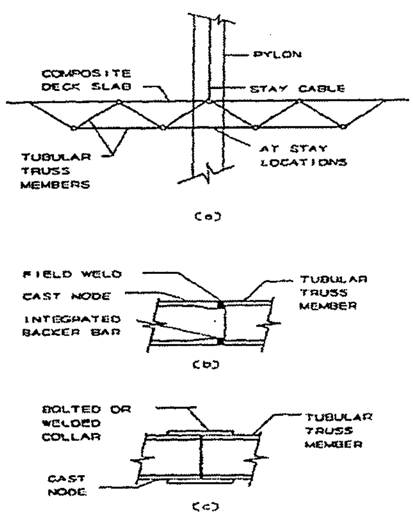
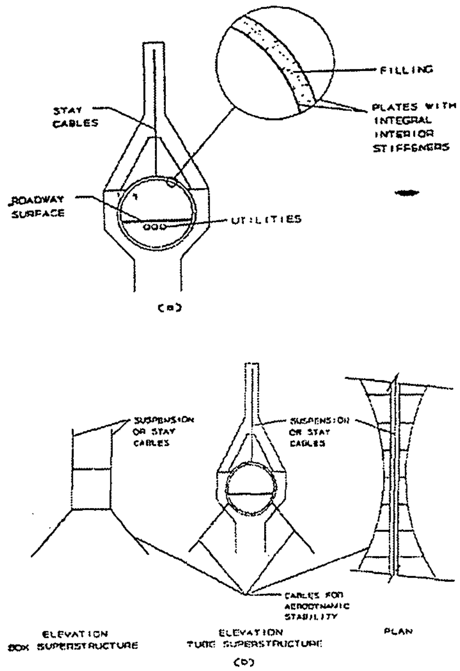
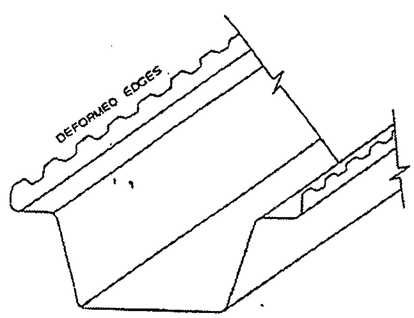
Concept	Description
14  Cold-formed, self-stiffened box girder	This concept utilizes the geometry of the box to provide the necessary out-of-plane stiffening for eliminating stiffeners. The use of prestressing and a concrete bottom slab can further enhance efficiency.
15  Tubular steel truss	This concept utilizes tubular sections to construct space or planar trusses and arches. Cast nodes are used to connect tubular members at joints by welding or by a split collar that can welded or bolted. Compression members can be filled with non-shrinking grout to increase stiffness. The truss can also be made composite with the concrete deck.

Table 11.1 (Contd). Innovative Concepts from FHWA Study [11.1].

Concept	Description
16  Cables for tubular cable-stayed long span bridges	This futuristic concept is intended for very large cable-stayed or suspension bridges. A large tube houses the bridge and utilities. Lateral support is provided by cables. The behavior of the section may be assumed to be similar to that of an airplane.
17  The use of deformed edges for composite action	Among new concepts for joining and fabricating elements is one in which deformed edges are utilized to develop composite action.

11.3 AISI Concepts

The AISI concepts were prepared by the Chicago office of J. Muller International (JMI) for short and medium span bridges. Several of the concepts utilize high performance steel in combination with prestressing to improve overall performance of traditional superstructures such as plate girders and box girders. In addition, innovative concepts such as space trusses used in the Roize Bridge (see p. 10.20) were also proposed. Complete information on the concepts may be found in Ref 11.2.

Unlike the FHWA concepts, those from AISI are much more detailed and were reviewed by industry for feasibility prior to their publication. The review took into consideration the viewpoint of fabricators, erectors, Federal Highway Administration, Departments of Transportation personnel, bridge engineers and academia.

Table 11.2 provides a very brief description of the AISI concepts. These were also reviewed by the same panel of bridge engineers that looked at the FHWA concepts. The comments were more favorable "*I believe this report provided enough details to define 'innovative' uses of steel and concrete composite structures. Any of these options can be further evaluated for suitability for larger spans. These spans can probably be made suitable up to the 300 ft span*" [11.3]. The two concepts that received guarded support for further consideration were (1) Split Single Steel Box Girder [p. 11.12.] and (2) Two-Span Twin Warren Truss [p. 11.13].

Comments for the split single box girder included (1) *very clean/aesthetic design* (2) *more expensive steel fabrication* (3) *need internal cross-bracing for erection/construction otherwise will need to erect on falsework* (3) *likely to need longitudinal bottom stiffeners as well to avoid excessive bottom flange thickness.*

Comments on the Warren Truss included (1) *scheme likely to have maintenance concerns because of numerous interfaces between steel and concrete* (2) *erection requires falsework or overhead gantry* (3) *very expensive as it requires a great deal of field assembly and labor* (4) *fatigue a possible concern at tube connections.*

References

11.1 Draft Interim Report prepared by Modjeski and Masters Inc. and ATLSS Center of Lehigh University. Presented by D. Mertz (1997) in paper "Innovative Bridge Design in High Performance Steel" at the Regional Seminar - High Peformance Steel: New Horizons in Steel Bridge Construction, Lincoln, Nebraska, October 6-7.

11.2 AISI (1996). "High Performance Steel Bridge Concepts", Chicago, November.

11.3 Canjura, N. (1998). Private Communication to R.Sen dated April 21.

Table 11.2 Innovative Concepts from AISI Study [11.2].

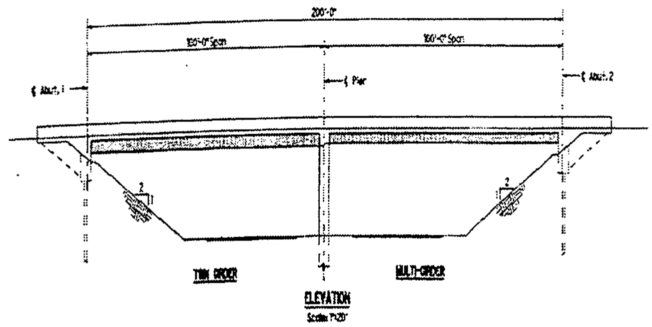
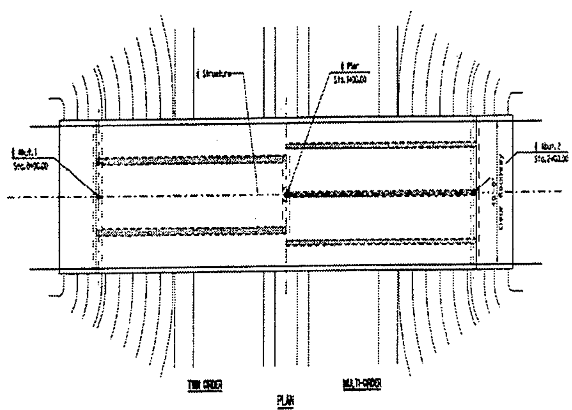
Concept	Description
1  Two span plate girder bridge system	For this plate girder concept, two alternatives are proposed (1) Prestressed two-girder system and (2) Multi-girder non-prestressed system. In either case, the spacing between girders is increased by haunching the concrete deck over each girder. Additionally, the concrete pier diaphragm is integral with the girders and piers. In the prestressed design, draped longitudinal post-tensioning is used to reduce the flange plate size. The deck is also post-tensioned longitudinally and transversely.
 Plan of two-span plate girder bridge system.	Two splice alternatives were investigated. The first uses a bolted top flange and a welded bottom flange making the connection continuous for live load, superimposed dead load and deck self weight. The second splice relies on composite behavior between the girder and the concrete deck. This connection is continuous for live load and superimposed dead load. The non-prestressed solution requires slightly more steel per square foot than the prestressed solution (12.1 psf vs 11 psf plus 0.412 psf for post-tensioning).

Table 11.2 (Contd.) Innovative Concepts from AISI Study [11.2].

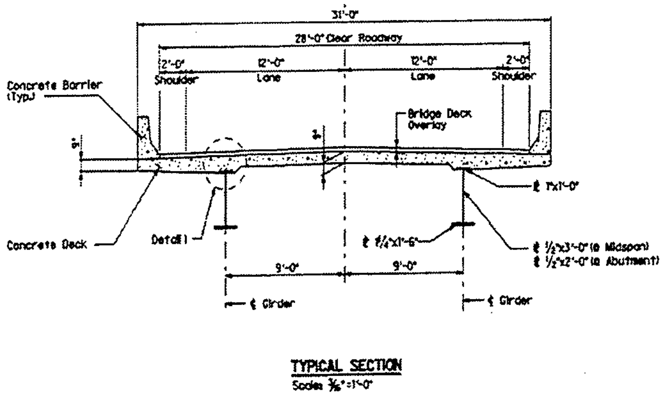
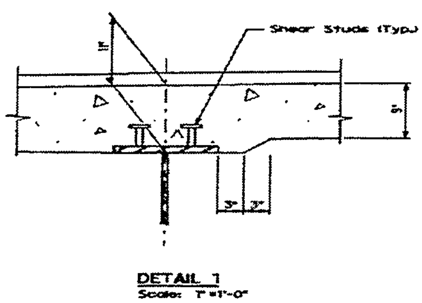
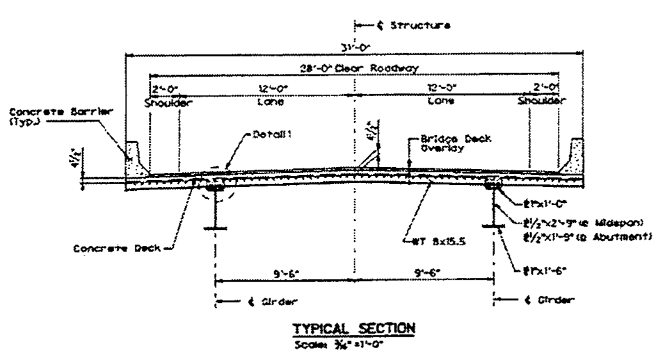
	Concept	Description
2	 Conventional concrete deck section  Connection details  Exodermic deck system	This is a single span composite plate girder bridge utilizing three different decks (1) conventional concrete deck (2) exodermic deck (3) grid reinforced deck. The girders are tapered to reduce the amount of steel needed. The simple span configuration can also be used for multi-span arrangements. Both the exodermic and grid reinforced concrete systems are significantly lighter than the conventional concrete deck.

Table 11.2 (Contd.) Innovative Concepts from AISI Study [11.2].

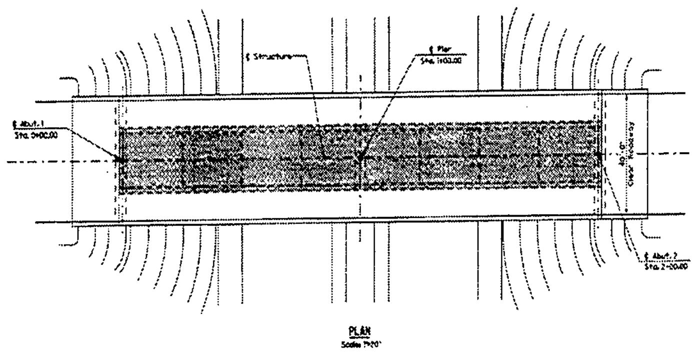
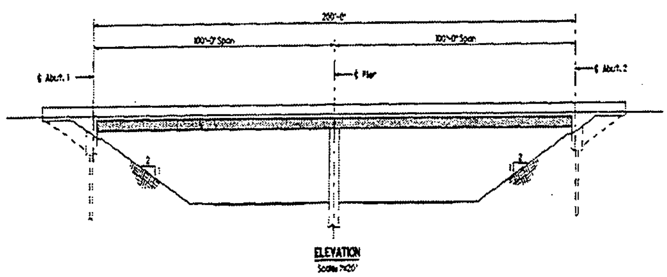
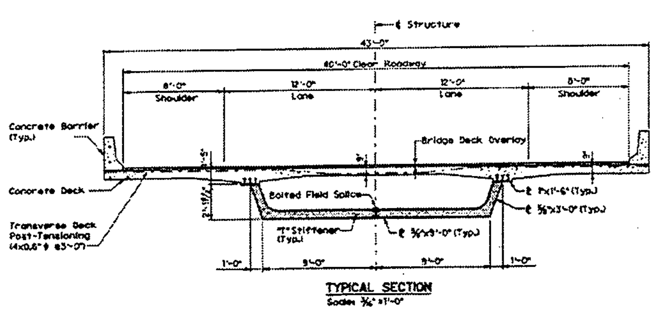
	Concept	Description
3	 Two-span split single box girder bridge system elevation.  Two-span split single steel box girder bridge system plan.  Two-span split single steel box girder bridge system section.	This concept consists of a transversely post-tensioned concrete deck that is composite with a single, split steel box girder. Single "T" sections are used as web stiffeners and distortion bracing. The box is assembled on site from two halves that are bolted together. The concrete deck is haunched over each top flange to increase the spacing between the boxes and the unsupported cantilevered deck. The concrete pier diaphragm is integral with the pier and the box girder. A concrete slab is cast inside the box over the pier to minimize the bottom flange thickness needed to resist large compressive forces.

Table 11.2 (Contd.) Innovative Concepts from AISI Study [11.2].

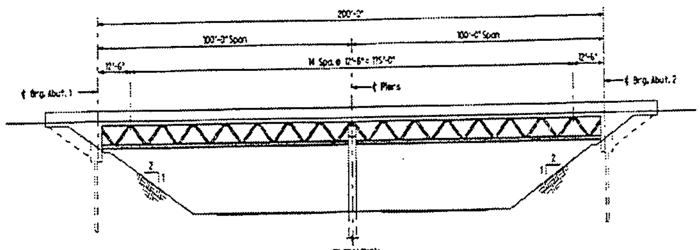
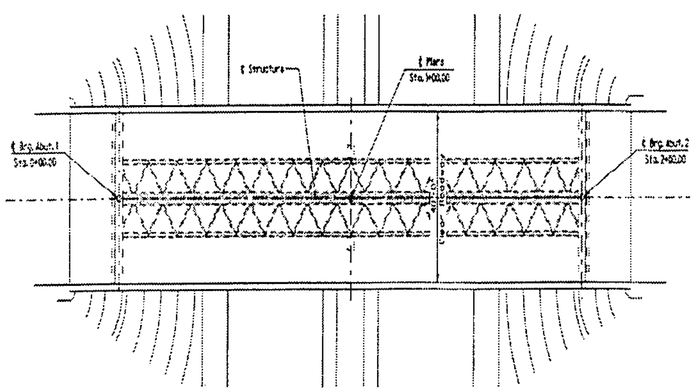
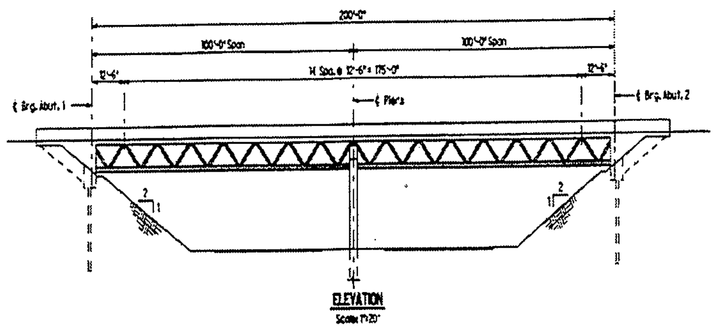
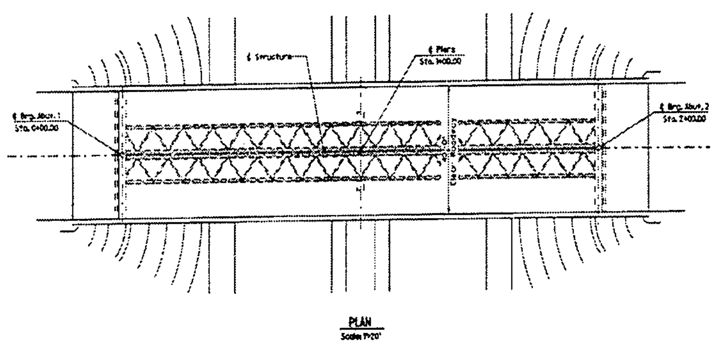
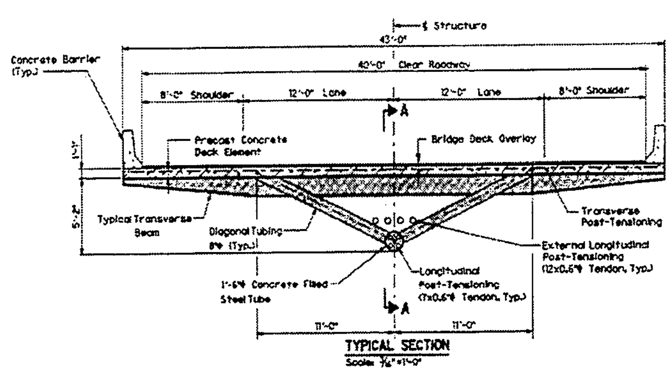
Concept	Description
4   Two-span twin warren truss bridge system plan	In this concept two parallel warren trusses support a cast-in-place transversely post-tensioned concrete deck connected to it by a longitudinally continuous voided steel plate and shear studs. The deck is also longitudinally post-tensioned by external prestressing located in the trusses. The inclined trusses are fabricated in 100 ft lengths and the diagonal and bottom chord members are steel tubes. The structure is continuous over intermediate supports and integral at the abutments. The span to depth ratio is greater than for a comparable girder structure and is therefore suitable where vertical clearance is not a critical consideration.

Table 11.2 (Contd.) Innovative Concepts from AISI Study [11.2].

Concept	Description
5  Two-span modular space truss bridge system elevation  Two-span modular space truss bridge system plan  Two-span modular space truss section	Despite the similarity in appearance to Concept #4, its construction is different. In this concept, precast, prestressed concrete deck panels are supported by a steel space truss that is continuous between abutments. A single steel tube filled with concrete and concentrically post-tensioned constitutes the bottom chord of the space truss. Hollow steel tubes make up the inclined warren type trusses that are welded to the steel tube and to I-shaped transverse floor beams at the top. The floor beams support the precast deck slabs and also constitute the formwork for the cast-in-place concrete closure pours between the deck panels. The precast, prestressed deck panels that equal the full deck width span between the transverse floor beams. Longitudinal post-tensioning is provided by draped tendons.

12. SUPERSTRUCTURE CONCEPTS FOR FLORIDA

12.1 Introduction

This chapter provides a brief summary of the process used to arrive at the double composite concept that is the subject of more detailed analysis and design in the next chapter. The review process is described in Section 12.2. Sections 12.3-12.4 contain information on additional concepts that were proposed in the course of the review. A critical analysis of all the short-listed concepts is presented in Section 12.5. A brief preview of the double composite concept is given in Section 12.6. Concluding remarks on the selection process appear in Section 12.7.

12.2 Review Process

The twenty one concepts presented in Chapter 11 were sent out for review during March/April 1998 to 15 organizations comprising both consulting engineers and contractors for their views on suitability. Only five responses were received with no clear consensus on a single concept that was right for Florida. Instead, lukewarm support was given to four - post-tensioned steel girder, tubular trusses, split single steel boxes (all from FHWA study) and a twin warren truss system (AISC/JMI study). The respondents expressed the opinion that the 200-600 ft span range was *too large for a single solution.*

To ensure that the views of experienced designers were adequately reflected in the decision making process, a meeting was held at USF on May 19, 1998 to review the AISC/FHWA and other concepts. The invited participants included some of the leading long span bridge designers in Tampa. The attendees, listed by organization, were *Mr. Theunis van der Veen, Mr. Paul Steijlen* (HDR), *Mr. Steve Stroh* (URS Greiner), *Mr. Richard Beaupre* (EC Driver), *Mr. Teddy Theryo, Mr. Jairam Hingorani, Mr. Victor Ryzhikov* (PB), *Mr. Jose Rodriguez, Mr Larry Sessions, Dr. Jose Danon* (FDOT), *Mr. Robert Clark, Jr* (Tampa Steel), *Mr. Jawahar Puvvala, Dr. Gray Mullins, Dr. Rajan Sen* (USF). Mr. Nelson Canjura (T Y Lin/DRC) was unable to attend.

In a three-hour meeting, all the concepts were re-viewed and additional ones discussed in the contextual framework of the present project. Clarifications were sought as to the type of structure intended, e.g. a two-lane ramp structure or four-lane intercoastal highway and the basis for having a 600 ft upper limit for the span range. The question of costs, maintenance and other issues such as the drawback of using prestressing steel in initial construction were also discussed.

The consensus of the meeting was the following:

(1) The study should focus on structures with spans in the 200-400 ft range. Larger spans tended to be one-off structures.

(2) The study should examine ways of enhancing existing concepts rather than look at brand new concepts, e.g. new materials. A double composite plate girder solution suggested by Mr. Steve Stroh and two truss solutions, one by Mr Theryo and the other by Mr. Steijlen were regarded favorably.

Following this brain-storming session, FDOT's program manager sent a draft letter on June 10, 1998 in which he proposed that the study focus on the following three concepts:

(1) *Warren Truss Concept*
(2) *Hybrid Truss Concept*
(3) *Twin Web PT Plate Girder with Spot Welded Epoxied Z Section*

The first of these concepts was proposed by the AISI/JMI study [Concept #4 in Table 11.2 on p. 11.13]. The second was proposed by Mr. Teddy Theryo of Parsons Brinckerhoff (see Section 12.3 below). The last was proposed by FDOT's Program Manager and is a modification of a FHWA concept [Concept #7 in Table 11.1 on p. 11.4]. In the letter it was suggested that conceptual designs and construction estimates be carried out for each concept using a 44 ft wide, 3-span bridge as a model. The proposed spans were 300 ft, 375 ft and 300 ft.

12.3 Hybrid Concept

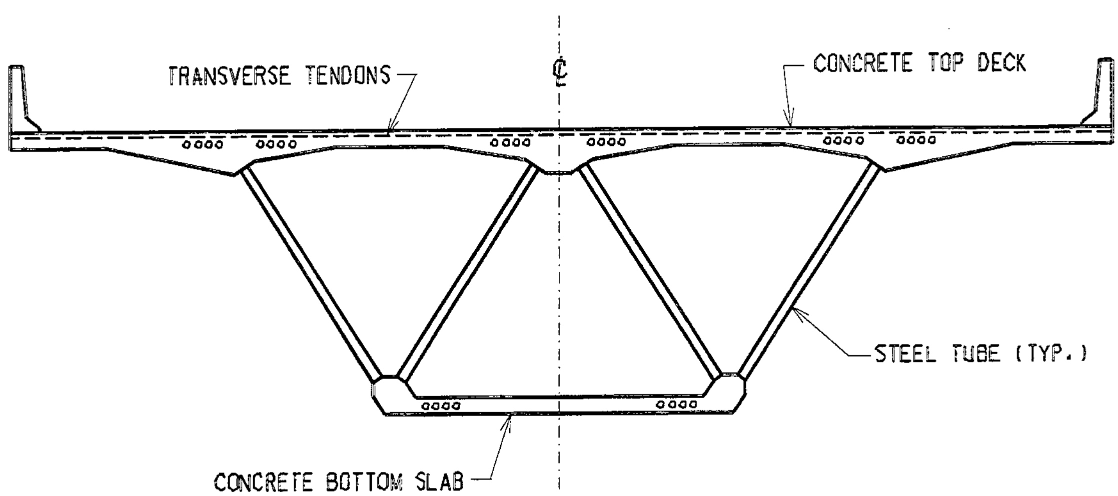


Figure 12.1 Hybrid Concept.

Box-sections, whether made of steel or concrete, are widely used for superstructure spans in the 200-800 ft range. Steel boxes are fabricated from thin plates and localized/ global buckling is a potential problem that is resolved by providing appropriate stiffeners. This inevitably increases its cost.

Concrete sections are more massive and are therefore less susceptible to buckling. Instead, they have relatively poor strength to weight ratio that limit their capacity to economically span greater distances. A hybrid box section in which the continuous concrete web is replaced by discrete steel tubes (see Fig. 12.1) can significantly reduce dead weight.

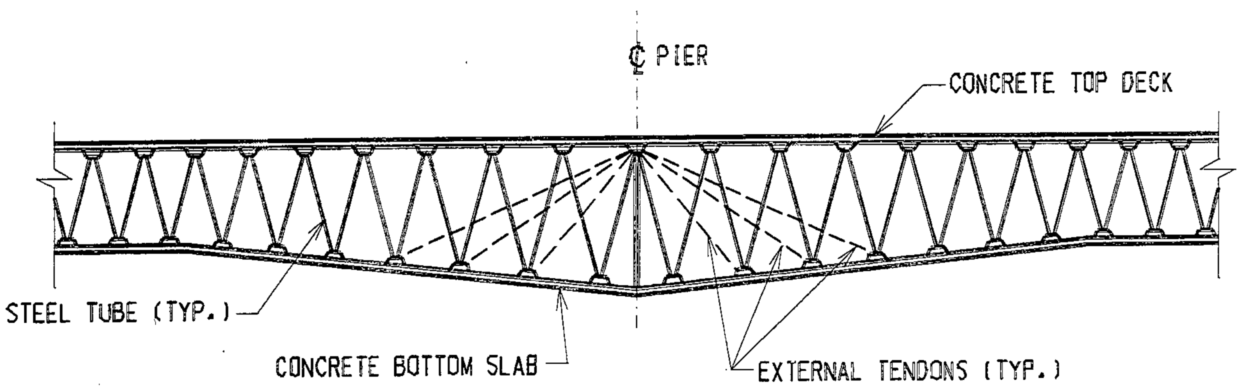


Figure 12.2 Schematic of Module Assembly.

The precast hybrid section would be cast as modules that would be erected using balanced cantilever construction with overhead gantries. Post-tensioning will be used to connect the modules (see Fig. 12.2).

The proposed concept is not new. Three new viaducts stretching across Enchinghen Valley near Boulogne in France were constructed using this concept in 1997 [12.1]. The intent is to modify and enhance the concept, particularly the steel tube/concrete flange connection, to make it more economical.

12.4 Twin Web Concept

Plate girders are easy to design and are widely used in bridges with spans in the short to medium range. The competitiveness of these structures would be improved if the webs could be made thinner without adding more stiffeners.

Corrugated webs have been used since the 1980's to achieve this objective (see Chapter 10). An alternative concept proposed by the FHWA study is to use two thin webs in conjunction with a core made of an unspecified material that would serve to provide increased resistance to buckling without adding to the weight of the structure [see p. 11.4].

FDOT suggested the replacement of the core material by a spot-welded, epoxied cold-formed Z section that would essentially serve as a spacer (see Fig. 12.3). As such, increased lateral stability would be attained with an off-the shelf material.

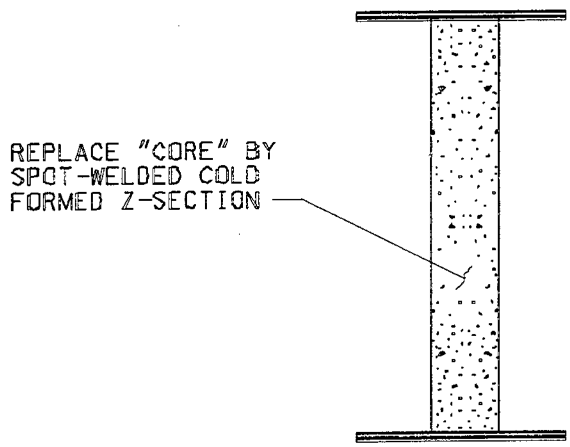


Figure 12.3 Twin Web PT Modified Concept.

12.5 Critical Analysis

The three concepts that arose from the review process have considerable potential. However, concerns were raised by USF relating to their suitability for this project both on technical grounds and also from the standpoint of the time frame and available Phase II funding. These are briefly outlined below:

Warren Truss Solution

The Twin Warren Truss solution was not considered suitable for long span bridges for the following reasons:

(1) The cross-section shown in Chapter 11[on p. 11.13] was not considered to be suitable for handling high negative support moments.

(2) The erection of the truss required falsework for the whole length of the bridge making it a less than an ideal solution for crossing intercoastal waterways or in urban areas.

(3) The numerous steel/concrete interfaces raised potential maintenance concerns.

(4) Costs could be considerable as it required a great deal of field assembly and labor.

Hybrid Box

The hybrid concept was considered unsuitable for this project for the following reasons:

(1) Detailing the connection between the steel tube and the deck was a major problem that would need to be experimentally and numerically investigated. The fabrication and particularly testing of the connections for such a complex structure could not be carried out with available funding for Phase II.

(2) The design is still experimental and is costly. According to ENR, the prototype structure was \$3.5 million higher than the lowest bid after Bouygues "*sacrificed potential profits by slashing about \$5 million from the price* [12.1]."

(3) The scheme envisages precast construction - the numbers of segments needed for spanning a single span of a intercoastal waterway may not be sufficient for this purpose.

(4) The concept had to be verified for its suitability for hurricane force winds. Wind tunnel testing needed to investigate this was not budgeted for in Phase II.

Twin Web PT

The proposed scheme utilized spot-welding and epoxying to cold formed Z sections to form the core. Lehigh University who have researched this type of construction were contacted. The following are some of the comments made by Prof. Richard Sause of Lehigh University in a message sent to one of PB's engineers [12.2]:

Our work showed that good structural performance can be achieved even with very thin face plates but durability of the epoxy/steel bond needs further investigation and welding of the composite web to the flanges poses some challenges that need special attention during fabrication.......I would not recommend spot welding without a thorough investigation because I believe spot welding could result in poor fatigue performance of the girder".

12.6 Double Composite Concept

In view of the important drawbacks in the concepts considered, a fourth concept presented by Mr. Steve Stroh at the meeting on May 19 was put forward for consideration. Again, this is not a new concept but has been used with considerable success in Germany. In essence, it involves the construction of a bottom concrete slab over the negative moment region to ensure double composite action as illustrated in Fig. 12.4.

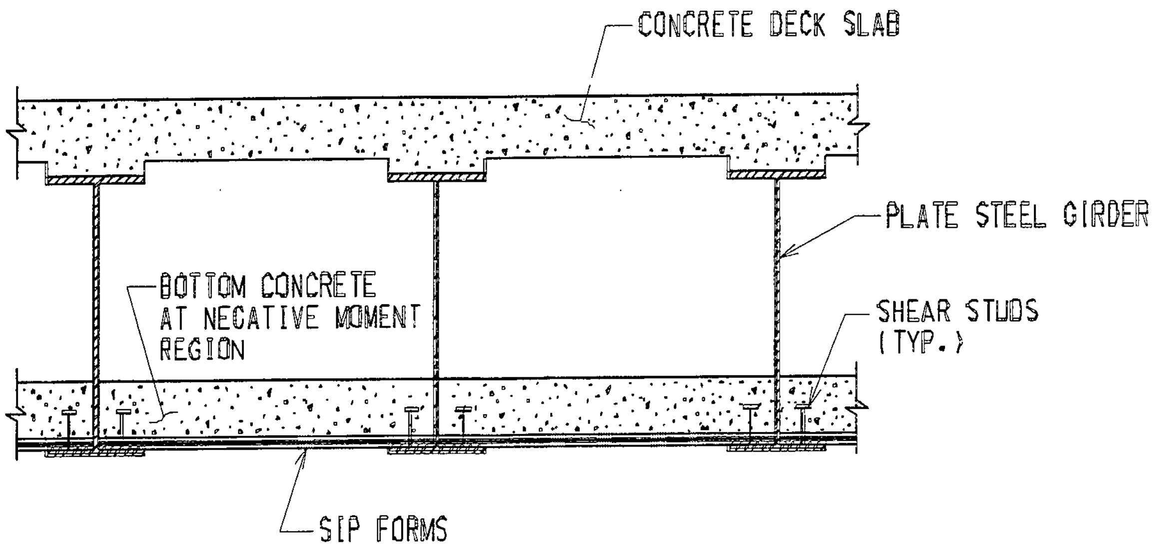


Figure 12.4 Double Composite Concept.

The double composite concept offers several benefits:

(1). It is not a radically new concept and is therefore easy to design. The vast majority of Florida's steel bridges are plate or box girder bridges so inspection and maintenance will not pose new problems.

(2) It can take advantage of existing LRFD provisions to reduce the number of cross-bracing frames needed that can lead to significant economies.

(3) Experimentation required in Phase II can be readily carried out since it will not involve complex elements or specialist fatigue or durability testing.

(4) The research findings can be immediately implemented (an important goal of this project) and is very likely to lead to success.

12.7 Conclusions

This chapter provides a very brief summary of the underlying basis for the selection of the double composite concept for this study. In the end, its choice was dictated primarily by the project's constraints. Given the two-year time frame and the $200,000 budget for the second phase, concepts that involved long term durability or fatigue testing or costly, elaborate fabrication, however meritorious, were automatically eliminated.

Many concepts that are promising may take years to evolve. The French who have been responsible for many innovative designs have not necessarily found them to be cost effective. The concept selected is not "new" but one that has enormous promise to make steel bridges more competitive in the state. In this connection, it is worth noting that with the construction of the Moore Haven Bridge with a main span of 321 ft [see p. 10.42], concrete girder bridges are gradually encroaching the span ranges where steel was traditionally more economical.

The ordinariness of the proposed concept makes it more likely to be routinely used in design. It is a economical structure that is not only simple to design but also easy to fabricate and erect. Economy was a very important consideration. As Mr. Robert Clarke (of Tampa Steel) eloquently stated at the meeting on May 19 "*Anything can be built. The only thing that limits our ability to perform is the customer's ability to pay*".

Apart from being an economical structure, double composite plate or box girders are also elegant structures that can enhance any site. Thus, they can be used with equal facility for crossing waterways or in urban environments.

References

12.1 "Owner, Builder Push Design Over Profit on French Bridge", ENR, June 23, 1997, p. 18.

12.2 Sause, R. (1998). E-mail on "I girder With Double Web" to N. Czaplicki. Forwarded to R. Sen on June 5.

13. STEEL BRIDGES WITH DOUBLE COMPOSITE ACTION

13.1 Introduction

This chapter provides results of an initial study of the double-composite bridge concept based on existing tools and design methods. Literature review on the application of the double composite concept is summarized in Section 13.2. The development of the concept is discussed in Section 13.3. In order to assess the concept, several plate girder and box girder designs were carried out using LRFD Specifications and the results compared against a "conventional" design utilizing composite action for only the positive moment region. The analysis and design for the conventional designs and their double composite counterpart are presented in Section 13.4-13.6. A comparative evaluation of these designs is presented in Section 13.7. Construction details are addressed in Section 13.8 and additional design considerations in Section 13.9. This section also includes comparisons for a hybrid girder. Concluding remarks are summarized in Section 13.10.

To prevent clutter, five appendices supplement the findings presented in this chapter. Appendix C contains details of design criteria and cost basis used in the study. The remaining four appendices provide detailed calculations relating to (1) Conventional Design (Appendix D), (2) Double Composite Plate Girder Calculations (Appendix E), (3) Double Composite Box Girder Calculations (Appendix F) and (5) Double Composite Hybrid Plate Girder Calculations (Appendix G).

13.2 Literature Review

The concept of a double-composite bridge is not a new idea, although it has seen limited application worldwide. A literature search was undertaken for this project, and only a few examples of double-composite bridges were identified. Saul [13.1-13.2] cites three examples of double-composite bridges in Germany, completed between 1992 and 1994. The double-composite bridge type was selected in there cases for a combination of economy, reduced deflections and aesthetics. Martinez-Calzon [13.3] indicates that the first double-composite bridge anywhere in the world was the Ciervana bridge in Spain in 1978, and cites several other examples of double-composite bridges built in Spain in the intervening years. Stroh [13.4-13.5] reports on a unique cable stayed double-composite bridge in Hong Kong, completed in 1995. No example of a double-composite design was found in the U.S., although the concept of a double-composite design was recognized in a 1996 report on innovative short and medium span bridge concepts prepared for the American Iron and Steel Institute [13.6].

13.3 Development of Double Composite Concept

13.3.1 General

In the U.S. the current design approach for steel girder bridges is to use a composite concrete deck. The deck acts compositely with the steel girder in the positive moment region of the span, helping to resist the top flange compressive stresses, and reducing the required cross sectional area of the steel top flange of the girder, and consequently result in cost savings. Shear connectors, such as shear studs, are provided to make the horizontal shear connection between the steel girder top flange and the concrete slab. In the negative moment region of the span the girder top flange is in tension, and following normal practice the tensile capacity of the concrete slab is neglected. Consequently, the concrete deck slab has no contribution to the flexural capacity of the girder.

There are two design approaches available for the design of the negative moment capacity of the girder. The girder can be designed non-composite in the negative moment region, with no shear connectors provided in this region. In this case the steel section alone carries all stresses. The second approach considers the reinforcing steel in the deck slab as part of the tensile steel resisting the negative flexure. In this case shear connectors are provided across the negative moment region of the span to engage the deck slab, with the tensile capacity of the slab neglected but the reinforcing steel considered. This approach can result in savings in top flange area, at the cost of the additional shear connectors in the negative moment region of the span and with the consequence of introducing a Category C fatigue detail for the top flange. The additional cost for the shear studs, and the introduction of the Category C detail are usually not too restrictive, and the most economical solution usually is to take advantage of the slab reinforcing steel to provide composite action in the negative moment region of the span.

In general, and from a cost versus capacity view, concrete is a less costly means of resisting compressive stresses. A simple calculation demonstrates this statement:

Say the unit cost of steel is \$2.20/kg and concrete is \$426/m^3, and use Grade 345 Steel and concrete strength of 28 MPa. The steel area to carry 1 MN force is:

$$1 \text{ MN} \div 345 \text{ MPa} = 0.00290 \text{ m}^2$$

and the resulting cost is:

$$0.00290 \text{ m}^2 \times 7850 \text{ kg/m}^3 \times \$2.20/\text{kg} = \$50.08/\text{m}$$

Assuming the concrete working at 0.85 f'c, the concrete area to carry 1 MN force is:

$$1 \text{ MN} \div (28 \text{ MPa} \times 0.85) = 0.0420 \text{ m}^2$$

and the resulting cost is:

$$0.0420 \ m^2 \times \$426/m^3 = \$17.90/m$$

As demonstrated for this simple example, the concrete slab is almost 3 times as efficient for carrying compressive forces, on a cost basis.

The basic concept of a double-composite design is to use a bottom concrete slab in the negative moment region of the span, replacing relatively costly steel with less costly concrete. As noted in the previous chapter, there are some additional benefits for a double-composite arrangement, namely:

- More efficient use of bottom flange steel by fully bracing the bottom steel flange by the slab and by allowing a compact design in the negative moment region.
- The elimination of some cross frames, afforded by the bracing of the bottom steel flange by the slab
- Favorable redistribution of moments within the girder due to the increased composite girder stiffened over the interior piers.

One of the principal goals of this study is the evaluation of the potential economy of the double-composite design concept, as compared to a more conventional composite girder design. The approach to this evaluation is to develop comparative designs for a "conventional" composite and double-composite girder and then make a cost evaluation.

For the purposes of this comparison, a bridge arrangement as shown in Fig. 13.1 was selected. This arrangement is for a three span continuous girder with spans of 65 m, 100 m and 65 m. This span length was selected as it is in the middle of the proposed study range of spans. A four girder arrangement with 3.7 meter girder spacing was used. The trial designs were limited to an interior girder location.

13.4 "Conventional" Plate Girder Bridge

The "conventional" design was initially optimized using the SIMON preliminary design software developed by US Steel Corporation. This program optimizes the web depth by making preliminary designs for a number of depth increments, then comparing the girder weight and cost. For our case, 17 trial designs were run with depth varying in 75 mm increments between 1975 mm and 3175 mm. Table 13.1 summarizes the resulting optimization, with the optimal arrangement having a web depth of 2500 mm. This 2500 mm web was selected for subsequent analysis.

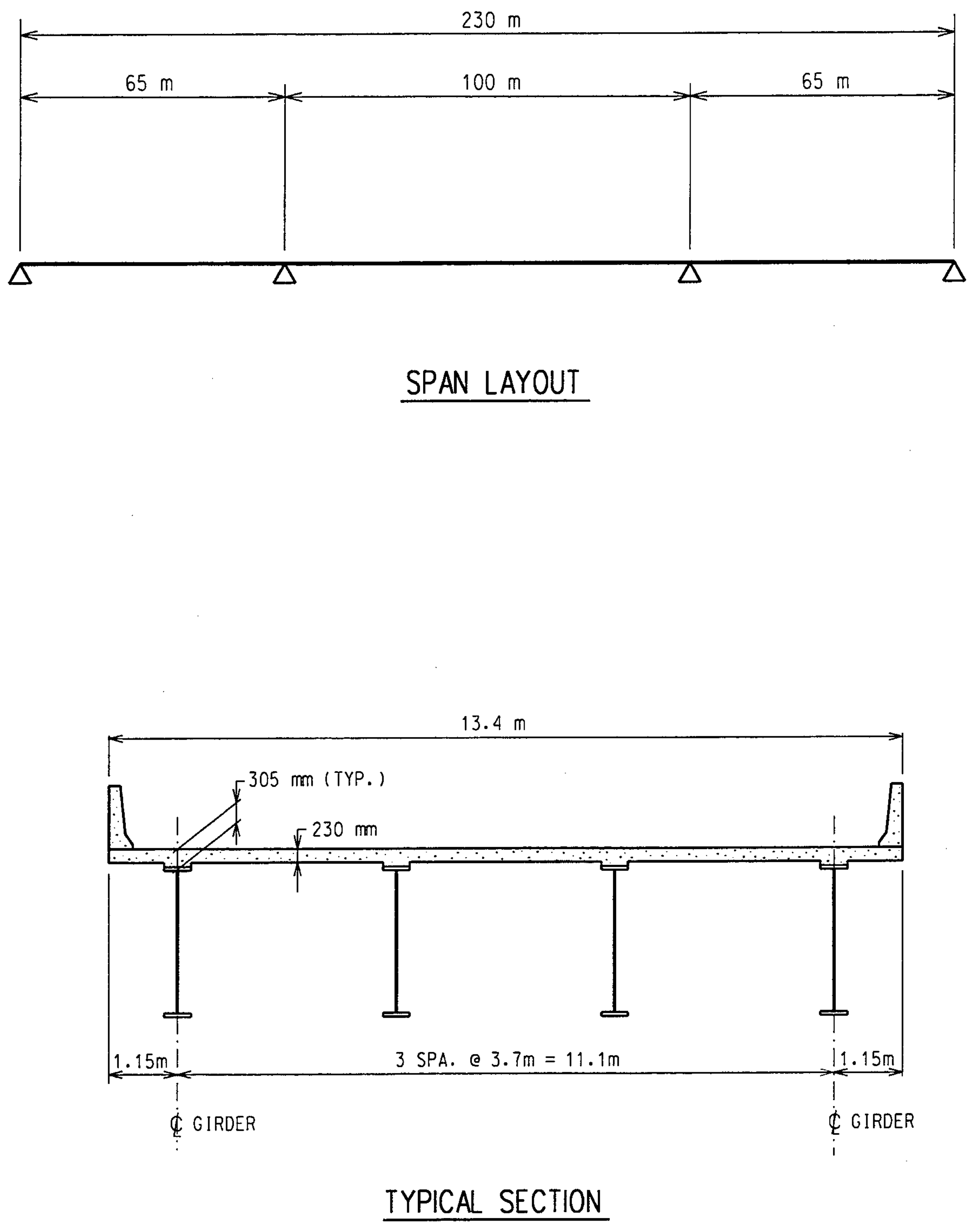


Figure 13.1 Bridge Arrangement for Comparison Studies.

Table 13.1 Web Depth Variation Analysis.

Trial	Web Depth (mm)	Girder Weight (Kg)	Index
1	1975	705625	1.08
2	2050	695319	1.06
3	2125	684505	1.05
4	2200	679130	1.04
5	2275	667616	1.02
6	2350	665773	1.02
7	2475	661142	1.01
8	2500	653170	1.00
9	2575	660733	1.01
10	2650	659762	1.01
11	2725	664574	1.02
12	2800	672183	1.03
13	2875	676106	1.04
14	2950	683969	1.05
15	3025	686394	1.05
16	3100	706960	1.08
17	3175	711491	1.09

The SIMON computer program is based on the AASHTO Standard Specifications, rather that the AASHTO LRFD Specifications, and was only used to establish an initial trial design and to optimize the web depth. The basis for the "conventional" design using the LRFD Specifications was to use the Washington State Department of Transportation (WSDOT) program Q ConBridge to generate the dead and live load moments, then to use a MATHCAD template to design the girder following the AASHTO LRFD Specifications and for subsequent iterations to optimize the design. The key design assumptions for the "conventional" plate girder design are as follows. Additional design criteria are shown in the Appendix C.

- A parallel flange girder is used with flange plates varied in response to moment demand. Four flange plate size steps are used in spans 1 and 3, and seven flange plate size steps are used in span 2.

- A constant thickness web is used, with transverse stiffeners as required for shear demand. No longitudinal stiffeners are used.

- The girder is taken as composite for both the positive and negative moment regions of the span. The composite concrete slab is considered in the positive moment region and the deck slab reinforcing steel is included in the composite action for negative moment region. One percent reinforcing steel is assumed in the slab in the negative moment region, in accordance with AASHTO minimum requirements.

- The resulting girder design is compact in the positive moment region and braced non-compact in the negative moment regions. A cross frame spacing of 6.5 meters is required in the negative moment region in order to meet the bottom flange bracing requirements for the section to qualify as a braced non-compact section.

- Strength and Service limit states were checked for the trial designs (Strength I and Service II). Fatigue or extreme event limit states were not considered for the purposes of this study.

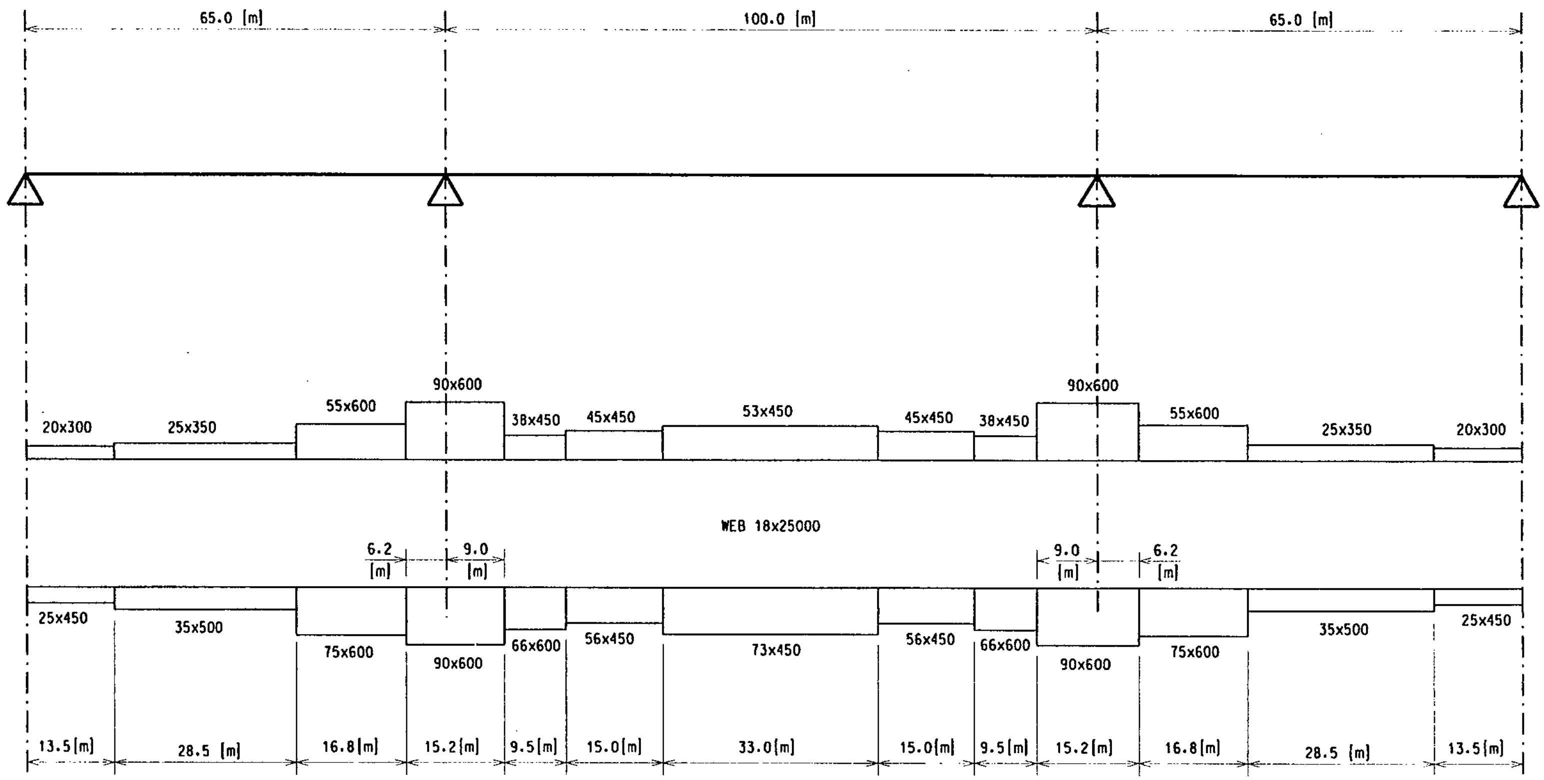
The assumed erection sequence for the "conventional" plate girder design follows normal practice for this type of construction. First, the steel girder is assumed to be erected in it's entirety. Next forms are placed for the deck slab then the deck slab concrete is placed, first in the positive moment regions, then over the piers. After the deck is cured the barrier and future wearing surface is placed. This erection sequence results in the following stress distributions:

- The dead weight for the steel girder, deck forms and deck slab concrete are carried by the bare steel (non-composite) section.

- The superimposed dead loads from the barriers and future wearing surface are carried on the composite section, meaning steel girder plus concrete slab in the positive moment regions and steel girder plus slab reinforcing steel in the negative moment regions. The composite action is based on a transformed section to include the deck slab with a modular ratio of "3n" where n is the ratio of the steel girder modulus of elasticity with the deck concrete modulus of elasticity, (the "3n" value is intended to account for long-term creep and shrinkage of the deck slab).

- Live load in carried by the composite section, based transformed sections with the value of "n".

Appendix D provides the MATHCAD output for the final optimization of the "conventional" design, and Fig. 13.2 shows the final plate sizes. The design was optimized such that the principal design parameters were within approximately five percent of their target values. Quantities were determined for the major superstructure elements and the cost estimate shown in Table 13.2 was developed.



NOTES:

1. PLATE SIZES IN MILIMETERS.
2. ALL PLATES GR. 345.

Figure 13.2 Girder Elevation – "Conventional" Plate Girder Design.

Table 13.2 Cost Estimate for "Conventional" Plate Girder Design.

Item	Units	Unit Cost	Quantity	Total Cost
Structural Steel (Gr. 345)	kg	$2.20	875,398	$1,925,876
Deck Concrete	m^3	$557.00	764.1	$425,604
Reinforcing Steel	kg	$1.00	91,690	$91,690
Pot Bearings	kN	$0.90	42,424	$38,182
Traffic Barrier	m	$72.00	460	$33,120
Total				$2,514,471

On a unit cost basis, the superstructure for the conventional alternate is $816/m² (or $75.85/ft²). The unit structural steel quantity is 284 kg/m² (or 58.1 lb/ft²).

13.5 Double-Composite Plate Girder Bridge

The double-composite design was developed in a manner similar to the "conventional" plate girder design. The design was based on the LRFD code, and used the WSDOT program Q ConBridge to generate the dead and live load moments. For the double-composite design, an EXCEL spreadsheet was used to design the girder and iterate to an optimal design. The choice of analysis tool, MATHCAD for the "conventional" design and EXCEL Spreadsheet for the double-composite design, was at the convenience of the analyst performing the calculations. Both programs were reviewed for consistency of approach to the design and accuracy.

The key design assumptions for the double-composite plate girder design are as follows. Additional design criteria are shown in Appendix C.

- A parallel flange girder is used with flange plates varied in response to moment demand. Four flange plate size steps are used in spans 1 and 3, and seven flange plate size steps are used in span 2 (the same arrangement as for the "conventional" plate girder design).

- A constant thickness web is used, with transverse stiffeners as required for shear demand. No longitudinal stiffeners are used.

- The girder is taken as composite for both the positive and negative moment regions of the span. The composite concrete deck slab is considered in the positive moment region. In the negative moment region, the bottom slab is taken as composite with the girder and the deck slab reinforcing steel is included in the composite action. For consistency with the "conventional" plate girder design, one percent reinforcing steel is assumed in the slab in the negative moment region. This assumption is further addressed later.

- The resulting girder design is compact in both the positive and negative moment region. A nominal cross frame spacing of 13 meters was used for the design.

- Strength and Service limit states were checked for the trial designs (Strength I and Service II). Fatigue or extreme event limit states were not considered for the purposes of this study.

The assumptions for the erection sequence for the double-composite plate girder design are important in that it determined how the forces are distributed in the girder, and therefore partially defines the economy of the system. The trial design assumes that first the steel girder is erected in it's entirety. Next, forms are placed for the bottom slab in the negative moment region and the bottom slab concrete is placed and cured. Next the deck slab forms are placed and the deck slab poured and cured, with a placement sequence similar to that noted for the "conventional" design. After the deck is cured the barrier and future wearing surface is placed. This erection sequence results in the following stress distributions:

- The dead weight for the steel girder, bottom slab forms and bottom slab in the negative moment region is carried by the bare steel (non-composite) section.

- The dead weight of the top slab forms and the top slab concrete is carried on the bare steel (non-composite) section for the positive moment region and on a composite section consisting of the steel girder and composite bottom slab (transformed by 3n) in the negative moment region.

- The superimposed dead loads from the barriers and future wearing surface are carried on the composite section, meaning steel girder plus concrete deck slab (transformed with 3n) in the positive moment regions and steel girder plus composite bottom slab (transformed with 3n) and top slab reinforcing steel in the negative moment regions.

- Live load in carried by the composite section, based transformed sections with the value of "n". In the positive moment region the composite section consists of the steel girder and top slab. In the negative moment region the composite section consists of the steel girder, the bottom concrete flange and the top slab reinforcing steel.

It is also noted that an alternate erection sequence could be used where the bottom composite slab is placed prior to erection of steel for the positive moment regions of the span. For this scenario, the girder field section over the pier would need to be shored during placement of the bottom concrete slab. For subsequent load cases, the girder section would be composite for all loads. This erection sequence would result in some savings in steel quantity at the expense of a somewhat more complex erection sequence.

Appendix E provides the EXCEL spreadsheet for the final optimization of the double-composite design plate girder, and Fig.13.3 shows the final plate sizes. Quantities were determined for the major superstructure elements and the cost estimate shown in Table 13.3 was developed.

On a unit cost basis, the superstructure for the double-composite alternate is $772/m^2 (or $71.76/ft^2). The unit structural steel quantity is 253 kg/m^2 (or 51.8 lb/ft^2).

Table 13.3 Cost Estimate for Double-Composite Plate Girder Design.

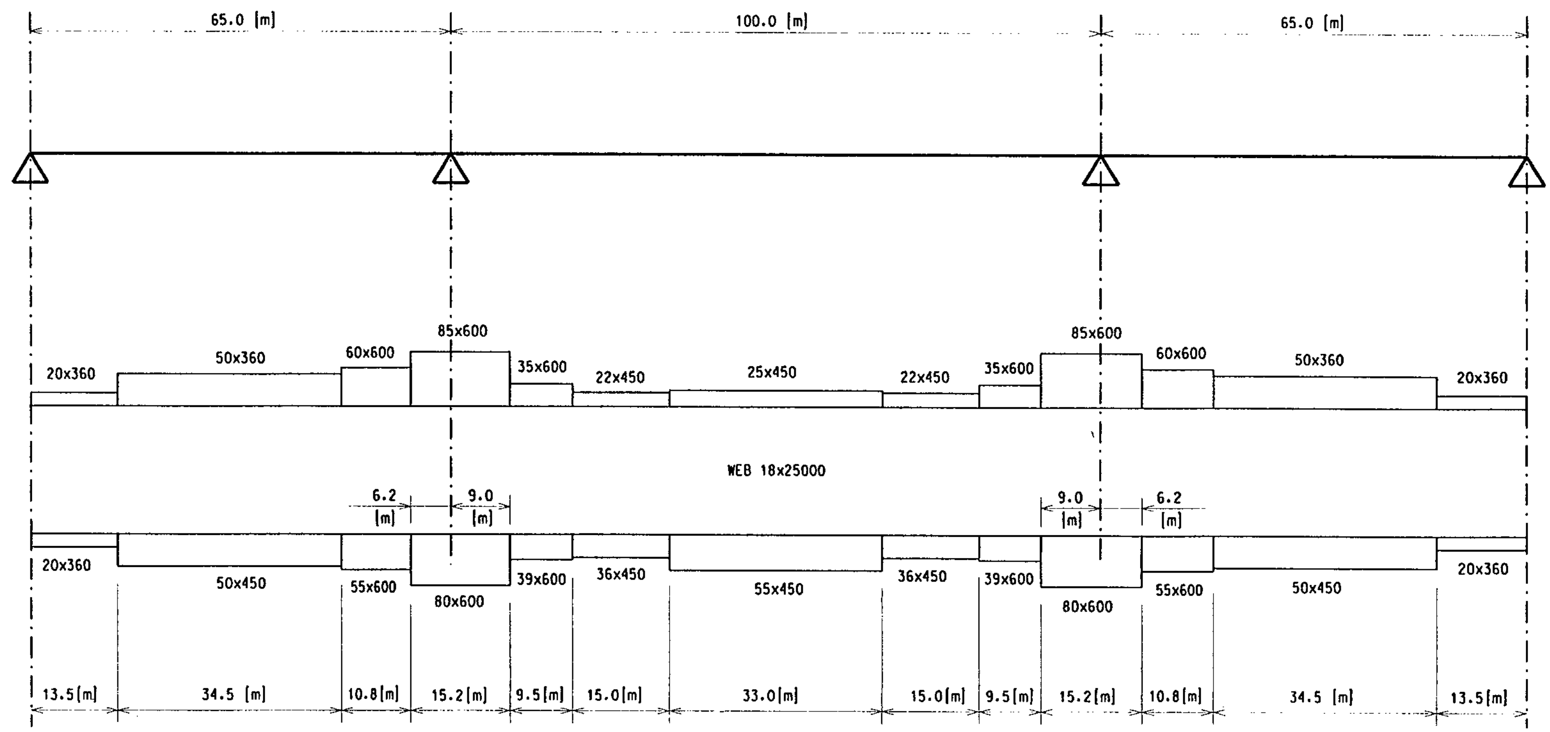
Item	Units	Unit Cost	Quantity	Total Cost
Structural Steel (gr.345)	kg	$2.20	780,309	$1,716,680
Deck Concrete	m^3	$557.00	764.1	$425,604
Bottom Slab Concrete	m^3	$400.00	129.8	$51,920
Reinforcing Steel	kg	$1.00	101,430	$101,430
Pot Bearings	kN	$0.90	56,064	$50,458
Traffic Barrier	m	$72.00	460	$33,120
Total				$2,379,211

13.6 Double-Composite Box Girder Bridge

As a variation on the double-composite concept, a double-composite box girder bridge was investigated. An advantage for this arrangement is that the bottom concrete slab can be placed directly in the steel box girder, without an additional form.

The superstructure arrangement for the double-composite box girder is shown in Fig. 13.4. A two girder arrangement was used for the trial design.

The double-composite box girder design was developed in a manner similar to the double-composite plate girder design. The design was based on the LRFD code, and used



NOTES:

1. PLATE SIZES IN MILIMETERS.
2. ALL PLATES GR. 345.

Figure 13.3 Girder Elevation – Double Composite Plate Girder Design.

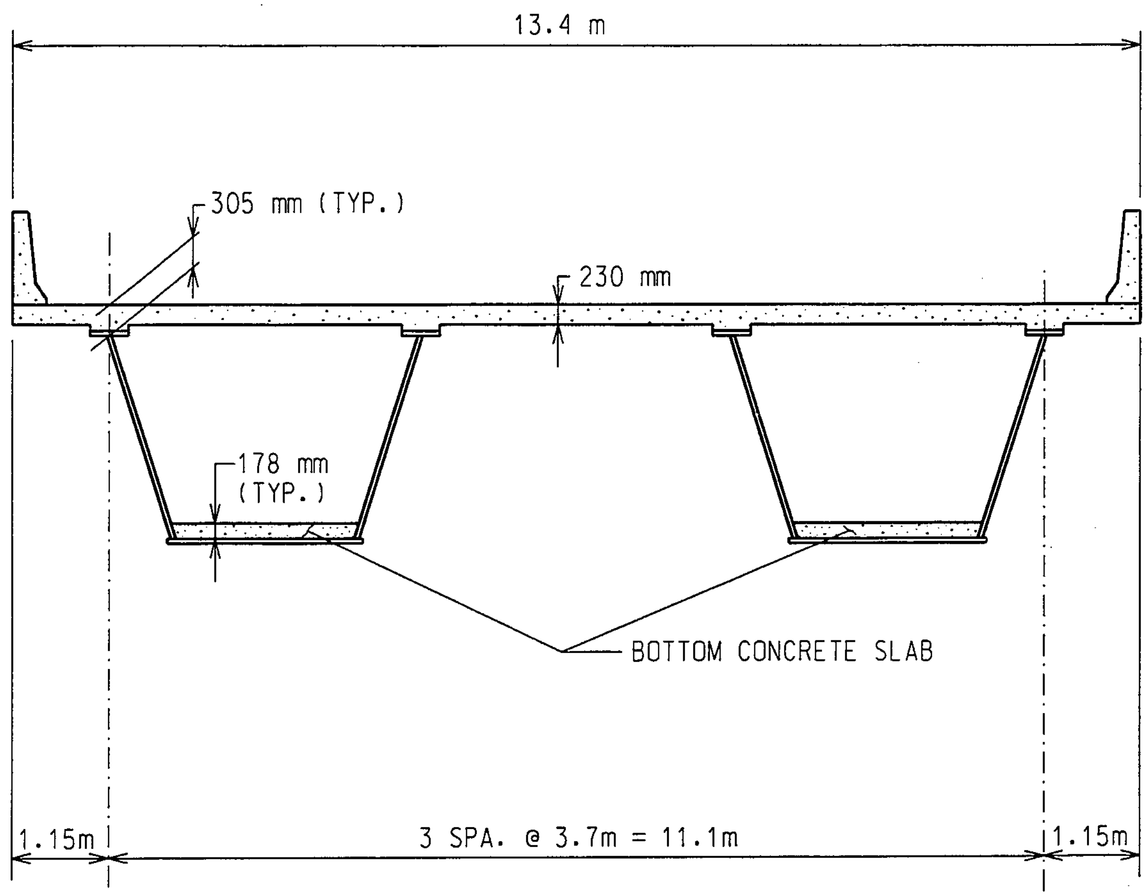


Figure 13.4 Typical Section – Double Composite Box Girder Design.

the WSDOT program Q ConBridge to generate the dead and live load moments. For the double-composite design, an EXCEL spreadsheet was used to design the girder and iterate to an optimal design.

The key design assumptions for the double-composite box girder design are similar to the double-composite plate girder design, as follows. Additional design criteria are shown in the Appendix C.

- A parallel flange girder is used with flange plates varied in response to moment demand. Four flange plate size steps are used in spans 1 and 3, and seven flange plate size steps are used in span 2 (the same arrangement as for the plate girder designs).

- A constant thickness web is used, with transverse stiffeners as required for shear demand. No longitudinal stiffeners are used.

- The girder is taken as composite for both the positive and negative moment regions of the span. The composite concrete deck slab is considered in the positive moment region. In the negative moment region, the bottom slab is taken as composite with the girder and the deck slab reinforcing steel is included in the composite action. One percent reinforcing steel is assumed in the slab in the negative moment region. This assumption is further addressed later.

- The resulting girder design is compact in both the positive and negative moment region. For the negative moment region, the AASHTO compactness requirements for bottom flange b/t ratio were not checked, since the bottom concrete slab is considered to fully brace the bottom steel flange.

- Strength and Service limit states were checked for the trial designs (Strength I and Service II). Fatigue or extreme event limit states were not considered for the purposes of this study.

The assumptions for the erection sequence for the double-composite box girder design are similar to the previously noted sequence for the double-composite plate girder design. The trial design assumes that first the steel girder is erected in its entirety. Next, the bottom slab concrete is placed in the box girder and cured. The deck slab forms are placed and the deck slab poured and cured, with a placement sequence similar to that noted for the plate girder design. After the deck is cured the barrier and future wearing surface is placed. This erection sequence results in the following stress distributions:

- The dead weight for the steel girder and bottom slab in the negative moment region is carried by the bare steel (non-composite) section.

- The dead weight of the top slab forms and the top slab concrete is carried on the bare steel (non-composite) section for the positive moment region and on a composite section consisting of the steel girder and composite bottom slab (transformed by 3n) in the negative moment region.

- The superimposed dead loads from the barriers and future wearing surface are carried on the composite section, meaning steel girder plus concrete deck slab (transformed with 3n) in the positive moment regions and steel girder plus composite bottom slab (transformed with 3n) and top slab reinforcing steel in the negative moment regions.

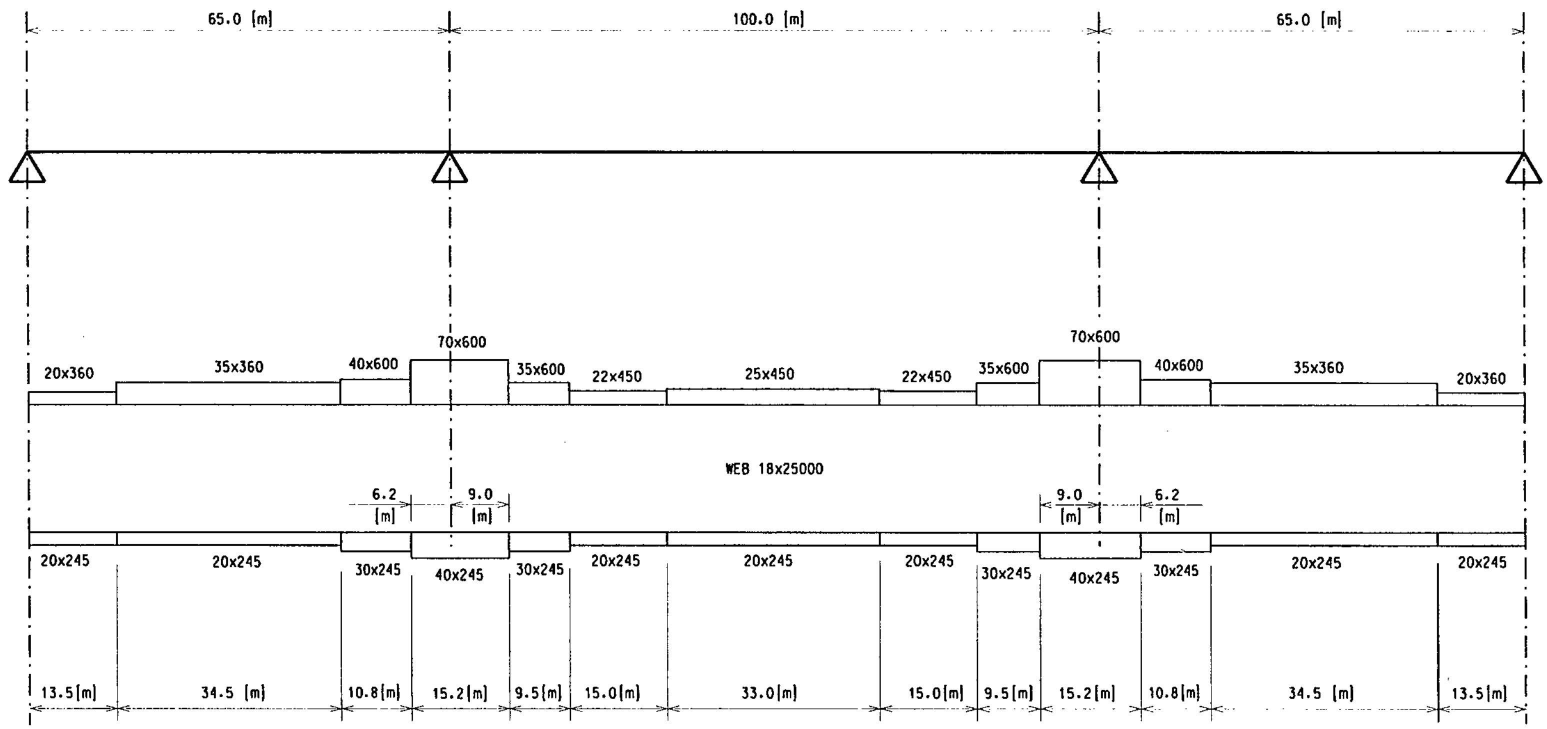
- Live load in carried by the composite section, based transformed sections with the value of "n". In the positive moment region the composite section consists of the steel girder and top slab. In the negative moment region the composite section consists of the steel girder, the bottom concrete flange and the top slab reinforcing steel.

It is also noted that an alternate erection sequence could be used where the bottom composite slab is placed prior to erection of steel for the positive moment regions of the span. For this scenario, the girder field section over the pier would need to be shored during placement of the bottom concrete slab. For subsequent load cases, the girder section would be composite for all loads. This erection sequence would result in some savings in steel quantity, at the expense of a somewhat more complex erection sequence.

Appendix F provides the EXCEL spreadsheet for the final optimization of the double-composite box girder design, and Fig. 13.5 shows the final plate sizes. Quantities were determined for the major superstructure elements and the cost estimate shown in Table 13.4 was developed.

Table 13.4 Cost Estimate for Double-Composite Box Girder Design.

Item	Units	Unit Cost	Quantity	Total Cost
Structural Steel (gr.345)	kg	\$2.40	793,546	\$1,904,510
Deck Concrete	m^3	\$557.00	764.1	\$425,604
Bottom Slab Concrete	m^3	\$330.00	58.1	\$19,173
Reinforcing Steel	kg	\$1.00	93,884	\$93,884
Pot Bearings	kN	\$0.90	54,373	\$48,936
Traffic Barrier	m	\$72.00	460	\$33,120
Total				\$2,525,227



NOTES:

1. PLATE SIZES IN MILIMETERS.
2. ALL PLATES GR. 345.
3. TOP FLANGE IN PAIRS.

Figure 13.5 Girder Elevation – Double Composite Box Girder Design.

On a unit cost basis, the superstructure for the double-composite box girder alternate is \$819/m^2 (or \$76.16/ft^2). The unit structural steel quantity is 257 kg/m^2 (or 52.8 lb/ft^2).

13.7 Comparisons and Evaluations

The trial designs show that the double-composite designs offer significant cost savings as compared to a "conventional" composite plate girder design. Based of the above described trial designs, direct savings indicated are summarized in Table 13.5.

Table 13.5 Comparison of "Conventional" and Double-Composite Designs.

Basis of Comparison	"Conventional" Plate Girder	Double-Composite Plate Girder	Double-Composite Box Girder	% Savings for Double-Composite Plate Girder Design
Weight of Girder [note 1]	2144 kN	1911 kN	1946 kN	12%
Unit Weight of Structural Steel	284 kg/m^2 (58.1 lb/sf)	253 kg/m^2 (51.8 lb/sf)	257 kg/m^2 (52.8 lb/sf)	12%
Total Superstructure Cost	\$2,514,472	\$2,379,211	\$2,525,227	6%
Superstructure Unit Cost	\$816/m^3 (\$75.85/sf)	\$772/m^3 (\$71.76/sf)	819/m^3 (76.16/sf)	6%

note 1: Weight of one girder, including cross frames (or one-half of a box girder)

The designs for the double-composite plate girder and double-composite box girder bridges yielded essentially equivalent cost designs. The box girder design required a slightly greater amount of structural steel, but had a lesser bottom slab concrete cost.

The indicated cost savings for the double-composite designs are a function of several factors. There is a 10% increase in stiffness for the negative moment section over the interior pier for the double-composite design, due to the increased stiffness of the composite bottom slab. As summarized in Table 13.6, this results in a favorable redistribution of moments for the double-composite design, due to the increased stiffness of the double-composite section over the pier attracting more moment. The decrease in moments at mid-span allow a smaller girder section for the positive moment region of the double-composite design, and the increase in negative moment over the pier is efficiently handled by the concrete bottom flange.

Table 13.6 Comparison of Live Load Moments.

Moment Location	"Conventional" Plate Girder	Double-Composite Plate Girder	Percent Difference
Max. Negative Moment	13,446 kN-m	15,900 kN-m	13%
Max. Positive Moment - Span 2	12,825 kN-m	11,300 kN-m	18%

The full bracing of the steel bottom flange by the bottom concrete slab allows a "compact" design in the negative moment region for the double-composite design. In addition to replacement of a portion of the negative moment bottom flange steel by less costly concrete, the compact design for the double-composite design in the negative moment region provides a more efficient use of the bottom flange structural steel.

The double-composite design allows elimination of some cross frames, afforded by the bracing of the bottom steel flange by the slab. For the "conventional" plate girder design a cross frame spacing of 6.5 m is required to achieve a braced non-compact section in the negative moment region of the span. For the double-composite design, the bottom concrete slab fully braces the bottom flange in the negative moment region, and therefore there is no specific requirement for cross frames. A nominal maximum spacing of 13 m is used for the cross frame spacing due to construction considerations. This results in a savings of 6 lines of cross frames for the double-composite design.

As previously demonstrated the bottom concrete slab has been shown to be more efficient, on a cost basis, for resisting compressive forces. However the bottom concrete slab is substantially heavier than the steel that is replace. This does not have a large affect on the overall girder moments since the bottom slab is only placed in the negative moment regions, and additional weight in this region have a small effect on the moments. However there is a substantial increase in the overall weight of the superstructure and the bearing reactions. For the subject trial designs, the superstructure weight for the conventional design is 33,800 kN and for the double-composite design the superstructure weight is 38,816, representing a 15% increase in dead load reaction. The live load reactions are essentially the same for the two alternates.

This additional dead load must be carried by the piers and foundations. However, depending on the design conditions, this additional dead load may not necessarily be a detrimental condition. For the case of piers governed by vessel impact forces, uplift of foundation elements is many times the limiting condition. In this case the additional dead load may result in a more efficient overall design. Typically bridge columns are governed by flexure, with the axial load well below the balanced point on the interaction diagram. An increase in axial load will actually allow a larger bending moment to be resisted by the column therefore, as before, the additional deal load may actually result in a more economical design. Obviously, the substructure design is dependent on the specific circumstances of each project. For this reason, no substructure comparisons have been included in the present study, and the cost evaluations have included only superstructure costs (including bearings).

13.8 Suggested Construction Details

Fig. 13.6 shows some possible construction detail for the bottom composite slab for the double-composite plate and box girder designs. For the plate girder alternate, preferably, stay in place slab forms can be utilized; either steel stay-in-place forms or concrete stay in place forms. However removable forms would also be possible. If concrete stay-in-place forms are used, the concrete forms should structurally be a part of the concrete slab. The bottom slab should be reinforced with nominal reinforcement for temperature and shrinkage.

The shear connection between the steel girder and the bottom concrete slab can be accomplished using conventional shear connectors, such as shear studs, following usual AASHTO design guidelines. For the trial design, 22 mm diameter studs placed in pairs at 200 mm spacing was required for the plate girder option.

An alternate detail for making shear transfer form the steel girder to the bottom slab could be to utilize rebar inserted through holed drilled in the girder web (Fig. 13.7). This concept is similar to the "perfobond" shear connectors described by Saul [13.1] and Leonhardt [13.7].

If the low point in the longitudinal profile of the bridge occurs in the region of the bottom composite slab, or if details are such that water could accumulate on the bottom slab, then drain holes should be provided at the low point for each section of bottom concrete slab.

13.9 Additional Design Considerations

13.9.1 Creep and Shrinkage

The behavior of concrete is time dependent. Concrete undergoes shrinkage, and under sustained load concrete exhibits creep. For a composite girder under sustained loads, this results is some redistribution of stress from the concrete to the steel.

For short term loading, such as live load, there is not sufficient time for creep to take place and an appropriate design methodology is to use a modulus of elasticity based adjustment of stiffness and stress. Typically the ratio of the steel to concrete modulus of elasticity (n) is used to transform the concrete to "equivalent" steel (concrete values are divided by n).

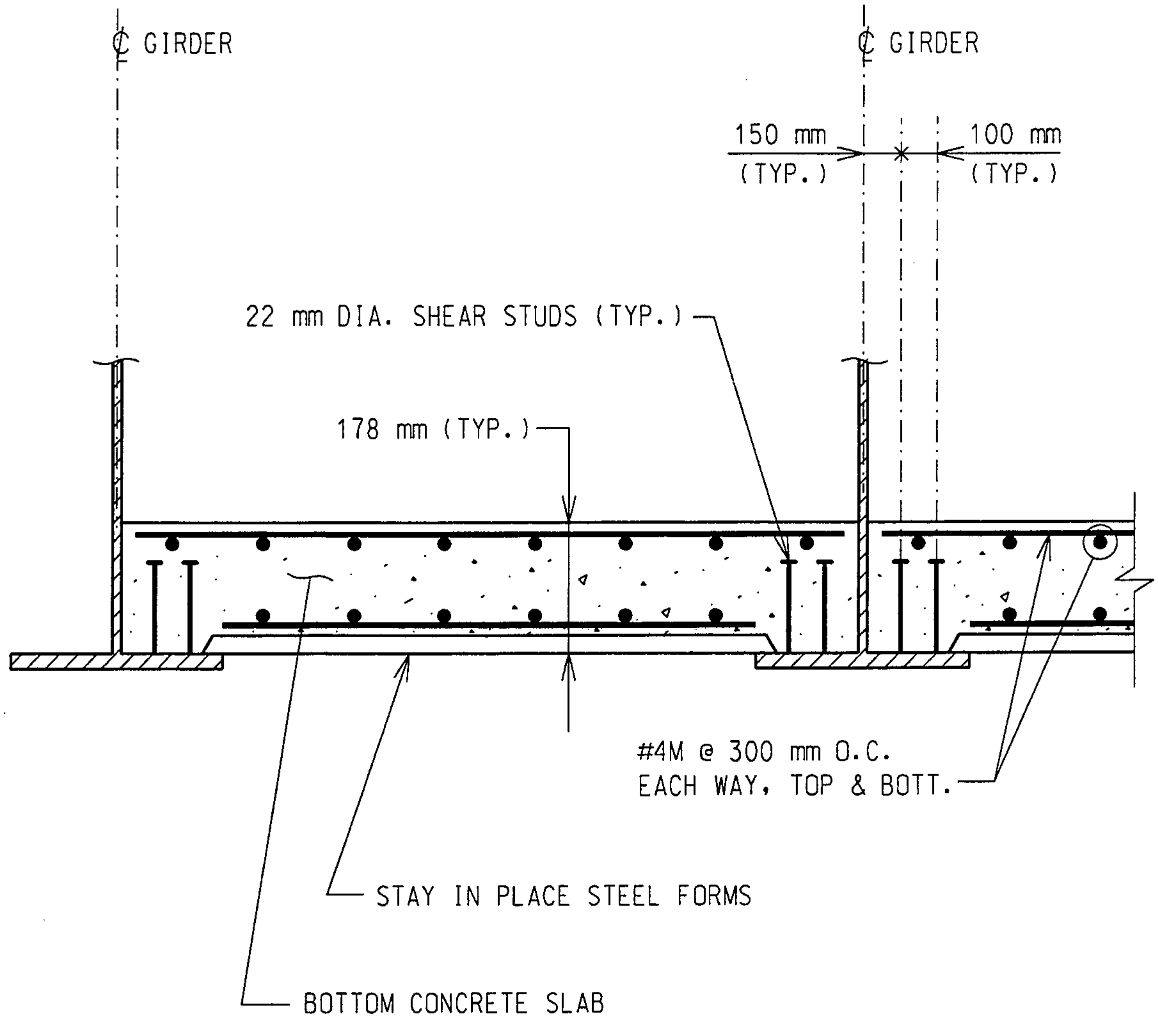


Figure 13.6 Construction Details for Double Composite Concept.

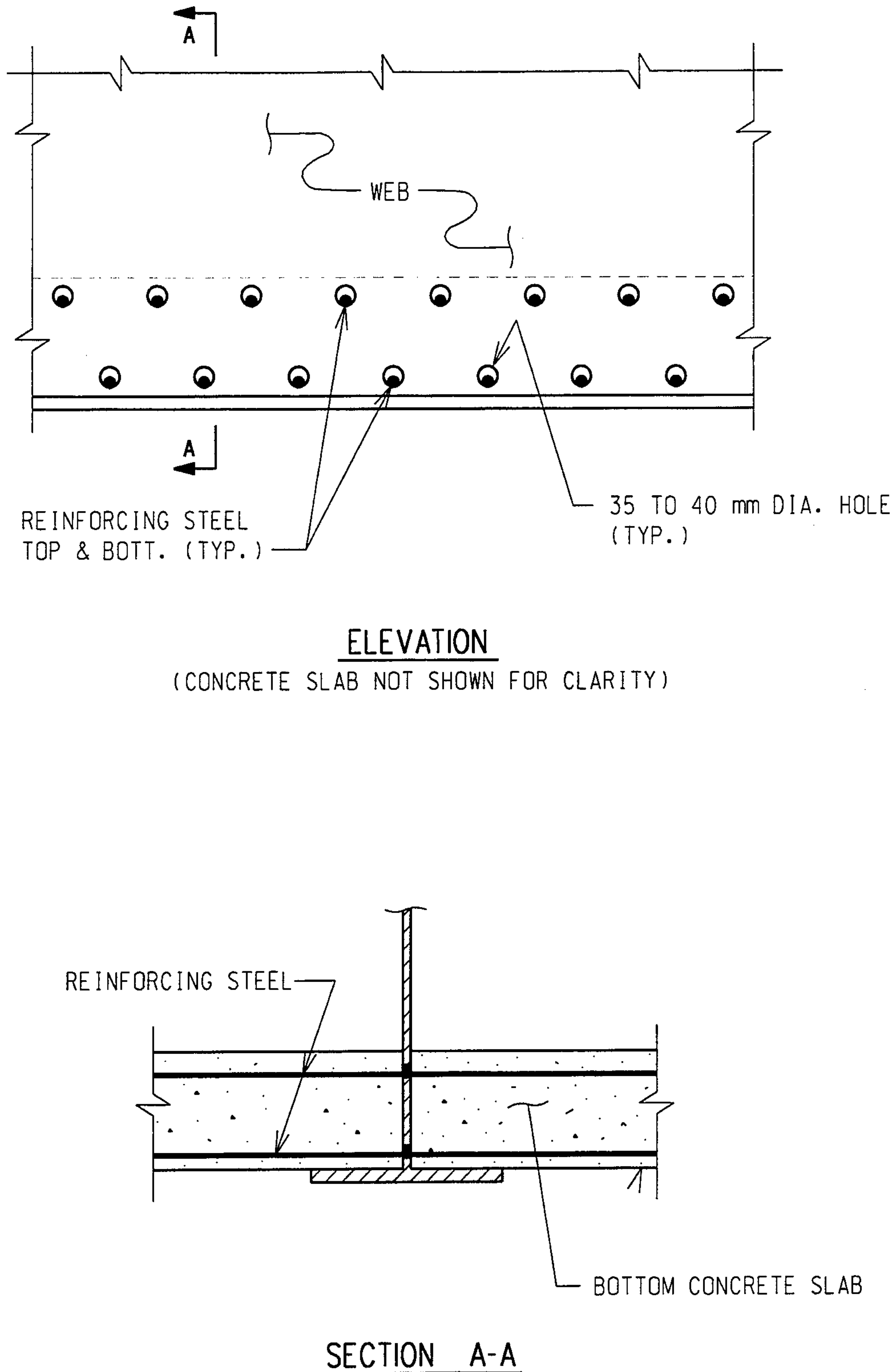


Figure 13.7 Detail of Alternate Shear Transfer Mechanism.

AASHTO recognizes the long term creep and shrinkage for sustained loading by using a *3n* ratio for transforming the concrete section properties. Based on the preliminary designs made as a part of this study, the stress levels developed in the bottom slab of the double-composite design are well within allowable limits at service limit state, and therefore the use of a *3n* modular ratio for all long term loading is considered appropriate for the double-composite designs.

13.9.2 Top Slab Serviceability

For negative moment conditions over the intermediate piers in a continuous girder, the concrete deck slab experiences tensile stresses. AASHTO recognizes this condition by requiring crack control reinforcing steel with an area equivalent to minimum of one percent of the deck slab area to be placed within the effective flange area; two-thirds of this steel to be placed in the top mat of reinforcing and one-third in the bottom mat. This provision is intended for "conventional" composite girder designs with only a composite top slab.

Saul [13.1] describes reinforcing details used for German double-composite bridges, providing 2.5% reinforcing steel over the intermediate piers, although the basis for selection of this reinforcing ratio is not given. Saul reports that fatigue testing was performed on a test section with the 2.5% reinforcing, and the section successfully sustained $2x10^6$ cycles and subsequently developed the full plastic moment.

For the trial designs, a reinforcing ratio of one percent was used and serviceability was evaluated in accordance with AASHTO limitations on reinforcing steel stress:

$$fs=\frac{z}{(d_cA)^{1/3}}\leq 0.6fy$$

AASHTO recommends a *z* value of 130 for severe exposure or 170 for moderate exposure. These z values correspond to approximate crack widths of 0.3 mm (0.013 in) for extreme exposure and 0.4 mm (0.016 in) for moderate exposure.

The above expression for *fs* was evaluated for the trial designs using a *z* of 130. This gave an allowable *fs* of 190 MPa (27.6 ksi). Based on a strain compatibility analysis, the maximum top slab reinforcing steel strain at service limit state was found to be 119 MPa (17.2 ksi) for the double-composite design, well within the AASHTO limits.

The reinforcing steel stress was also evaluated for the "conventional" design using similar assumptions, and a nearly identical stress of 121 MPa (17.6 ksi) was computed. Therefore at service limit state the performance of the deck for the double-composite design is expected to perform similar to a conventional composite design.

However, there is a difference in the behavior of the double-composite design at the ultimate limit state. For a compact section there are three basic criteria that must be met. A b/t ratio for the compression flange to address local buckling of the flange, a d/t_w ratio for the web to address buckling of the web and a requirement of lateral bracing to address global buckling. For a conventional plate girder design, the requirements for a compact section generally cannot be economically met, either due close spacing of cross frames for global stability or with regard to the web thickness requirements for web stability in excess of that required for shear. Therefore the negative moment region of conventional composite girder designs are typically in the elastic range at strength limit state, either braced non-compact or unbraced non-compact.

With regard to a double composite design, the bottom concrete slab lowers the neutral axis such that web slenderness requirements for a compact section can be met and the bottom flange fully braces the compression flange for global stability. Therefore a compact section is readily achieved for the double composite design.

At the strength limit state the compact section is utilized to develop the full plastic moment capacity of the section, for comparison against the ultimate moments. Under this stress state, the concrete deck slab is not participating in the flexural resistance, but it is cracked and under the steel plastic strains. Coincident with this load condition the slab must resist local flexural and shear stresses due to wheel loads on the deck.

The AASHTO code does not address this stress interaction; in fact shear is not typically checked in deck slabs. Typical composite deck slabs have performed well without specific design requirements for interaction between the longitudinal strains resulting form composite action and local transverse flexural and shear demands. However as previously noted, the longitudinal design of the girder is typically in the elastic range. For the proposed double composite design, where a plastic stress distribution is possible, this interaction needs further consideration.

13.9.3 Application of High Performance Steel

In addition to the savings afforded by the double-composite concept, the possibility exists for use of high performance steels. Using a hybrid design with gr. 485 flanges and a gr. 345 web, and following a similar design procedure as the previously described double-composite plate girder design, the quantities and costs shown in Table13.7 were developed.

Compared with the homogeneous double-composite plate girder design using all gr. 345 steel, the above represents nearly an 8% percent cost savings. Compared with the homogeneous "conventional" girder design, the cost savings for the hybrid double-composite design is nearly 14 percent. Particularly if the double-composite plate girder design is to be compared with an alternative bridge system, such as concrete I-girders, the use of a hybrid design with higher strength gr. 485 flanges should be considered.

Table 13.7 Cost Estimate for Hybrid Double-Composite Plate Girder Design.

Item	Units	Unit Cost	Quantity	Total Cost
Structural Steel (gr.345)	kg	$2.20	273,451	$601,592
Structural Steel (gr.485)	kg	$2.42	389,947	$943,672
Deck Concrete	m^3	$557.00	764.1	$425,604
Bottom Slab Concrete	m^3	$400.00	129.8	$51,920
Reinforcing Steel	kg	$1.00	101,430	$101,430
Pot Bearings	kN	$0.90	54,917	$49,425
Traffic Barrier	m	$72.00	460	$33,120
Total				$2,206,763

13.10. Conclusions

The double-composite bridge concept appears to be a viable and economical bridge alternative that may provide a truly competitive bridge option in the 60 to 120 meter span range for Florida's Bridges. Based on the present studies, the double-composite steel bridge is shown to have a potential cost savings as much as 14 percent as compared with a conventional plate girder design.

With the exception of a few details, the design of a double-composite girder bridge is adequately covered by the existing AASHTO design code. The available computer software for steel design is not structured to accommodate a double-composite design, however, as demonstrated in the trial designs prepared as a part of this study, it is relatively straight forward to use existing software to perform the analysis (using software such as Q ConBridge, or any other steel design software with appropriate adjustment of girder stiffnesses) then using MATHCAD or a spreadsheet to perform the design.

It is recommended that the double-composite design concept be further pursued. Specific recommendations are contained in the next chapter.

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14. RECOMMENDATIONS FOR PHASE II

14.1 Introduction

The comparative designs presented in the previous chapter established the viability of the double composite bridge concept and demonstrated the savings that were realizable if it were used. This chapter defines the scope of the studies proposed for Phase II that will help to further refine and develop the concept.

14.2 Objectives and Scope

The focus of Phase I was primarily to develop an appropriate innovative concept. Consequently, a single trial span configuration was investigated with no special attempt to optimize the concept nor to address specific design, detailing or construction limitations, e.g. limits on girder segment lengths and weights and their implication on the design of prototype structures.

In keeping with the objectives set out in the original RFP, the goal in the second phase of the study is to specifically address the above outstanding issues through appropriate design studies, laboratory tests and numerical analyses.

The principal objectives may be summarized as follows:

1. To conduct appropriate parametric design/analysis to optimize the performance of doubly composite plate girders.

2. To conduct appropriate numerical analyses and laboratory tests to establish new design criteria.

3. To conduct scale model tests on a three-span continuous hybrid plate girder bridge to evaluate the performance of the proposed criteria and also to examine the feasibility of alternative shear transfer methods for the bottom slab.

4. To develop model examples that can be used for the design of a prototype structure.

Brief details of the studies that are proposed to achieve each of the above objectives are discussed in the following sections.

14.2.1 Design Optimization

In the trial designs, only interior girders could be considered. The design of the exterior girder will also need to be carried out. This has an unsymmetric section since the bottom slab only extends on one side for aesthetic considerations.

The comparative evaluations considered a single span configuration because of time limitation. It is recommended that further trial designs be prepared for conventional plate girder designs versus double composite designs over a range of applicable spans, e.g. 200-400 ft. It would also be instructive to make comparisons with concrete girder designs over similar span lengths to establish economic competitiveness.

In the trial designs, the girders were spaced at 12 ft. For longer span structures, the spacing could be increased for greater economy. It has been suggested that a spacing of up to 20 ft could be used in conjunction with high strength concrete and transverse prestressing of the deck slab [14.1]. Other measures proposed that can lead to more economical designs, such as the use of integral piers or post-tensioning may also need to be examined.

The goal of the design studies is to establish spacing limits that offer the greatest economy when conventional slabs or prestressed slabs are used. The largest spacing will be used as the basis for model bridge tests proposed (see 14.3.3).

14.2.2 Design Criteria

An important reason why the double composite concept results in greater economy is the continuous bracing of the compression flange in the negative moment region that allows the use of compact sections. Compact sections can not only reach their full plastic capacity but their increased stiffness makes them more efficient in re-distributing moments at the strength limit state. In contrast, in conventional composite girder designs, the sections are always non-compact so that their stress levels are in the elastic range at the same limit state.

A critical design issue not addressed by the AASHTO specifications is the ability of the top slab to transfer the shear forces from local wheel loads while under simultaneous plastic strain from composite action with the girder. Under this condition, the slab is heavily cracked in the transverse direction and it is not known whether the superimposition of the wheel load strains will result in a safe condition for the top slab. Additional longitudinal reinforcing steel, above the 1% steel normally provided, may be required to remedy this situation.

In order to develop a rational design procedure it is necessary to conduct both numerical analyses and laboratory tests. Initially, a parametric study will be conducted using three dimensional non-linear analysis but because of the inherent complexity of the problem, the finite element solution needs to be calibrated and verified experimentally. The aim of the testing will be to provide data that will be used to establish a relationship between the important parameters such as girder spacing and the amount of top slab reinforcement.

A schematic arrangement of the proposed test, identical to that in Ref. 14.2, is shown in Fig. 14.1. In the set-up, a segment of a continuous bridge between points of inflection (taken as 0.2L, where L is the span) will be tested. The support reaction is represented by a single upward load applied at the middle. Downward loads are applied at the cantilevered ends to simulate the negative moments at the support. These loads will be increased until the girders reach their plastic state. At these loads, the deck slab will be severely cracked in the transverse direction [14.2] as shown in Fig. 14.1. Additional downward wheel loads will then be applied between the girders to determine the slab's ability to support these forces. The wheel loads will be increased to their design value to determine the failure mechanism.

Tentatively, it is assumed that two different girder spacings will be tested with the top slab reinforcement initially limited to 1% to be consistent with current AASHTO specifications. Should shear failure result, an additional test will be conducted using a greater amount of reinforcing steel to assure a ductile failure. Transverse post-tensioning of the slab may also be considered.

The results of the tests will be used to calibrate the finite element model. Parametric studies will be carried out using the refined model so to a develop a simplified design criteria. This will be utilized in the design of the scale model bridge and verified during testing described in the next section.

14.2.3 Model Bridge

To further refine the double composite concept, service and ultimate limit state tests will be carried out on a three-span continuous girder model. In the model, the spacing of the girders will be kept as large as possible as this leads to the greatest economy. The top slab reinforcement will be designed in accordance with criteria already developed. In addition, alternate methods for transferring shear (other than stud connectors) will be investigated, e.g. reinforcing steel inserted through holes in the web (see schematic in Fig. 14.2).

A hybrid model using high performance steel flanges will be tested since the design trials indicated that it yielded the greatest benefit. The central span will be 20% larger than the side spans as shown in Fig. 14.2. It is expected that the girders will be spaced approximately 20 ft apart in the prototype structure. Consideration will be given to using a transversely prestressed deck slab. The exact dimensions and scale to be adopted for the model will be based on the findings of design studies (14.3.1) and discussions with the Florida Department of Transportation.

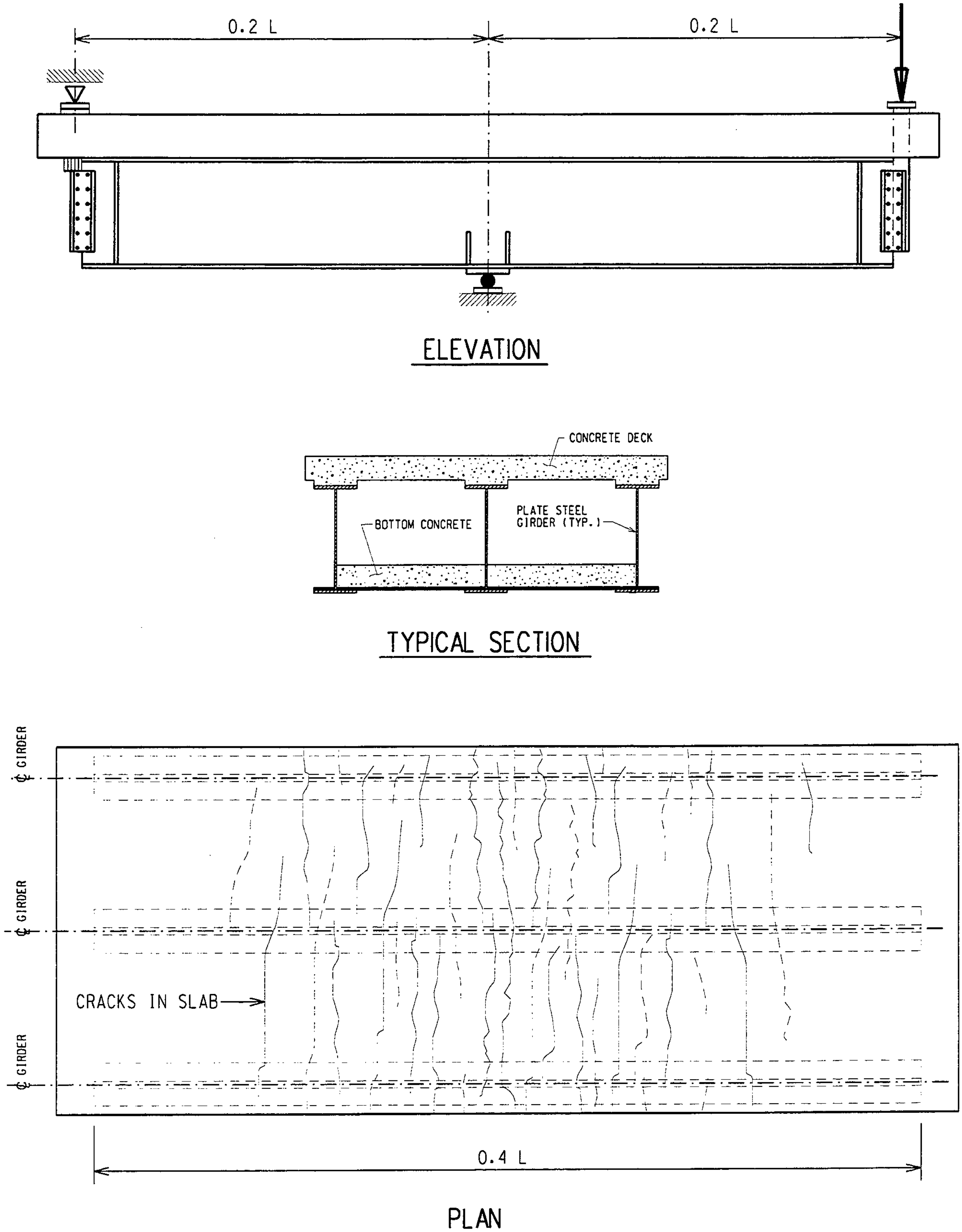

Figure 14.1 Test Setup for Top Slab Reinforcement Criteria.

The choice of a three-span configuration was dictated by its facility to allow investigation of two alternative shear transfer mechanisms at the two supports as shown schematically in Fig. 14.2. The model will be tested to evaluate load distribution characteristics and also its elastic and ultimate response. In the ultimate load tests, the shear transfer mechanism of the top slab over the supports will be automatically evaluated for two cases - one for each of the two supports. Appropriate analyses, e.g. grillage or finite element, will be carried out to correlate with the experimental results.

14.2.4 Model Examples

On the basis of the findings of the study, detailed design examples will be prepared to facilitate use of the double composite concept for typical applications. These examples will illustrate the design of interior and exterior girders and will highlight differences with those for conventional design. To ensure clarity, they will be circulated for comment prior to the publication of the final report.

14.3 Recommendations for Prototype Development

Based on favorable consideration of the double-composite bridge concept, it is recommended that a demonstration project be selected and a prototype structure be constructed. This study will serve several purposes:

- It would validate the economy and constructability of the double-composite bridge concept.

- It would allow instrumentation and testing of the design assumptions, in particular the performance of the deck slab in the negative moment region related to crack control, and allow confirmation of long term creep and shrinkage performance.

Additionally, the possibility exists for such a project to secure discretionary funding as a demonstration project, supplementing Florida's transportation allocations. And in the long term, development of an alternate bridge type for the 60 to 120 meter span range should foster competition and lower costs for Florida's bridges.

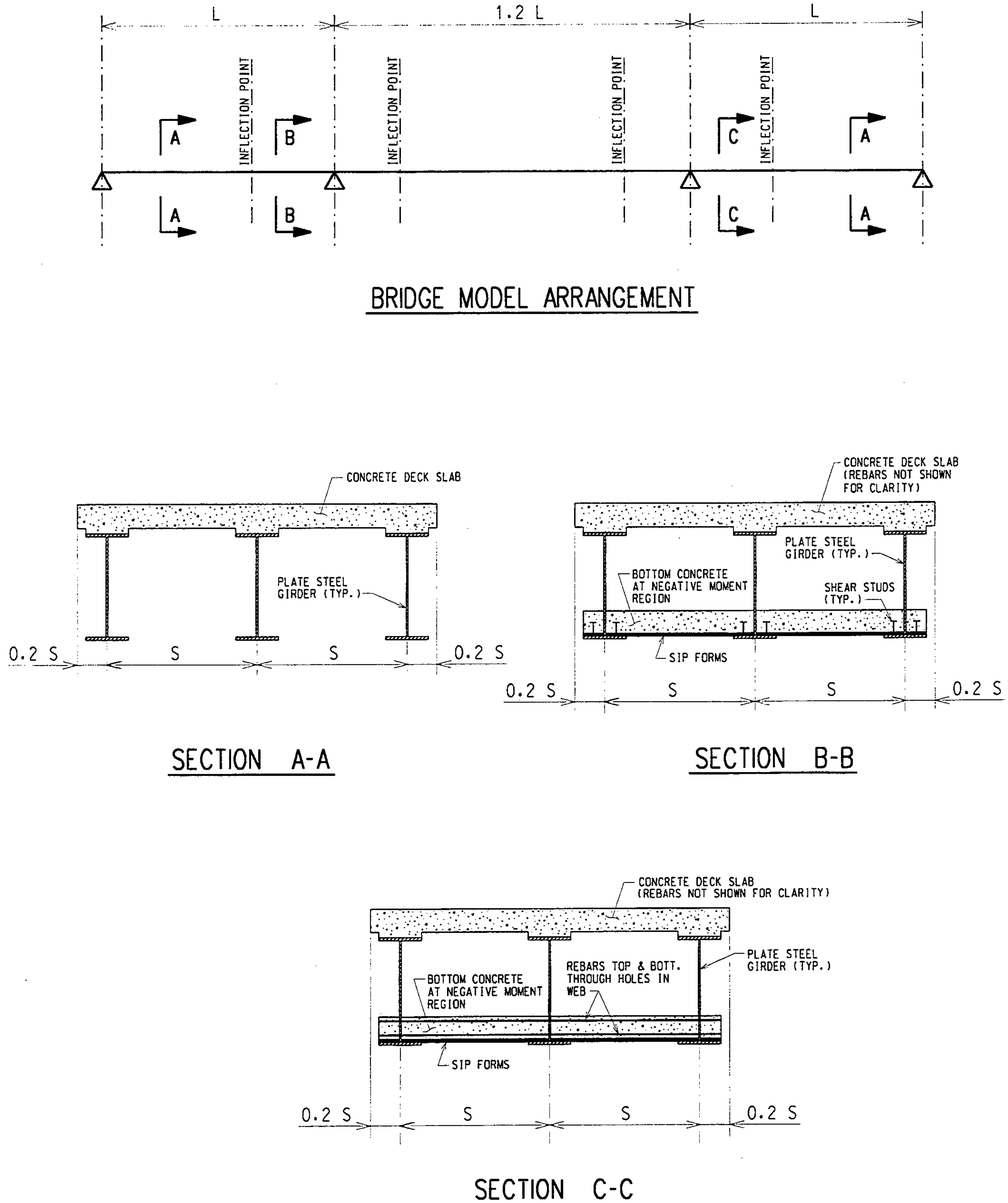

Figure 14.2 Proposed Bridge Testing Detail.