



National Library of Canada

Cataloguing Branch
Canadian Theses Division

Ottawa, Canada
K1A 0N4

Bibliothèque nationale du Canada

Direction du catalogage
Division des thèses canadiennes

NOTICE

The quality of this microfiche is heavily dependent upon the quality of the original thesis submitted for microfilming. Every effort has been made to ensure the highest quality of reproduction possible.

If pages are missing, contact the university which granted the degree.

Some pages may have indistinct print especially if the original pages were typed with a poor typewriter ribbon or if the university sent us a poor photocopy.

Previously copyrighted materials (journal articles, published tests, etc.) are not filmed.

Reproduction in full or in part of this film is governed by the Canadian Copyright Act, R.S.C. 1970, c. C-30. Please read the authorization forms which accompany this thesis.

**THIS DISSERTATION
HAS BEEN MICROFILMED
EXACTLY AS RECEIVED**

AVIS

La qualité de cette microfiche dépend grandement de la qualité de la thèse soumise au microfilmage. Nous avons tout fait pour assurer une qualité supérieure de reproduction.

S'il manque des pages, veuillez communiquer avec l'université qui a conféré le grade.

La qualité d'impression de certaines pages peut laisser à désirer, surtout si les pages originales ont été dactylographiées à l'aide d'un ruban usé ou si l'université nous a fait parvenir une photocopie de mauvaise qualité.

Les documents qui font déjà l'objet d'un droit d'auteur (articles de revue, examens publiés, etc.) ne sont pas microfilmés.

La reproduction, même partielle, de ce microfilm est soumise à la Loi canadienne sur le droit d'auteur, SRC 1970, c. C-30. Veuillez prendre connaissance des formules d'autorisation qui accompagnent cette thèse.

**LA THÈSE A ÉTÉ
MICROFILMÉE TELLE QUE
NOUS L'AVONS REÇUE**

PRESTRESSED CONCRETE BRIDGES
ERECTED BY THE SEGMENTAL CANTILEVER METHOD

Vassily Verganelakis

A Major
Technical Report
in
The Faculty
of
Engineering

Vassily Verganelakis 1977

Presented in Partial Fulfillment of the Requirement for the
degree of Master of Engineering at Concordia University,
Montréal, Quebec, Canada.

September, 1977

ABSTRACTVassily VerganelakisPRESTRESSED CONCRETE BRIDGES
ERECTED BY THE SEGMENTAL CANTILEVER METHOD

Although the cantilever construction method applied to prestressed concrete bridges has been used for more than two decades in Europe and elsewhere, its application in North America is very recent.

In fact only during the present decade, in Canada as well as in the United States, a move in this direction started with some good results.

The purpose of this thesis is to give a general idea of this type of structure and the related construction and design problems.

After a brief historical review of the evolution of concrete bridges, the description of the method and the choice of the statical system are given. In the following chapters, the different construction methods are examined and the design problems are discussed.

In conclusion, some examples of such bridges are given showing the different construction methods used today throughout the world.

ACKNOWLEDGEMENTS

The author wishes to extend his gratitude to Dr. M.S. Troitsky, Professor of Civil Engineering at Concordia University for his valuable advice, criticism and suggestions. His direct supervision and encouragement are greatly appreciated.

He is also indebted to Dr. F.A. Gerard, Professor of Civil Engineering at Concordia University for his kind help.

Thanks are also extended to Mr. Francis Boulva, Consulting Engineer for giving the author the opportunity to conceive and analyse the prestressed concrete bridge at Grand'Mère.

Finally a special credit should be given to Mr. Norman Kadanoff, Consulting Engineer, for his valuable linguistic aid and Miss Josette Mathieu who typed the manuscripts.

TABLE OF CONTENTS

	<u>Page</u>
ABSTRACT	III
ACKNOWLEDGEMENTS	IV
LIST OF FIGURES	X
1. INTRODUCTION	1
2. HISTORY	4
3. DESCRIPTION	15
3.1 Building with Blocks	15
3.2 Advantages	18
3.3 Field of Application	21
4. STATICAL SYSTEM AND PRESTRESSING STEEL LAYOUT	27
4.1 Lengthwise Statical System	27
4.1.1 Bridges with One or Two Hinges in Each Span	28
4.1.1.1 Bridges with One Hinge	28
4.1.1.2 Bridges with Two Hinges	30
4.1.2 Continuous Deck	33
4.1.3 Cable-Stayed Bridges	38
4.2 Transverse Statical System	39
4.3 Prestressing Steel Layout	43
5. CONSTRUCTION	54
5.1 General	54
5.2 Construction by Cast-in-Place Segments	57
5.2.1 General	57

TABLE OF CONTENTS CONT'D

	<u>Page</u>
5.2.2 Construction Methods	59
5.2.2.1 Symmetrical Cantilevering	59
5.2.2.2 Scaffolding and Cantilevering	61
5.2.2.3 Cantilevering and Cable-Stay	65
5.2.2.4 Cantilevering with an Auxiliary Beam	67
5.2.3 Construction Phases of a Segment	67
5.3 Construction by Erecting Prefabricated Segments	74
5.3.1 General	74
5.3.2 Prefabrication Methods	76
5.3.2.1 Precasting on a Prefabrication Bench (Long Line Method)	76
5.3.2.2 Precasting in Stationary Moulds (Short Line Method)	83
5.3.3 Transportation and Erecting of the Segments	88
5.3.4 Joints and Epoxy	96
5.4 Comparison of the Two Discussed Methods	100
5.5 Deck Stability During Construction	102
5.5.1 Permanent Anchorages	102
5.5.2 Temporary Anchorages (General Case)	105
5.6 Construction by the Pushing Method	106
5.6.1 General	106
5.6.2 Description	107

VII

TABLE OF CONTENTS CONT'D

	<u>Page</u>
5.6.2.1 Casting and Advancement of the Deck	107
5.6.2.2 Reinforcement	111
5.6.2.3 Sliding Apparatus	114
5.6.3 Applications	116
5.6.4 Advantages of the System	118
6. DIMENSIONING AND DESIGN	122
6.1 Dimensioning	122
6.1.1 Choice of the Spans	123
6.1.2 Substructure	123
6.1.3 Depth and Moment of Inertia of the Girder	128
6.1.4 Transverse Deck Section	134
6.1.4.1 Bottom Slab	134
6.1.4.2 Top Slab	136
6.1.4.3 Webs	137
6.1.4.4 Haunches	137
6.1.4.5 Shear Keys	138
6.1.4.6 Segment Length	138
6.2 Design	140
6.2.1 Design for the Construction Period	146
6.2.1.1 Substructure	146
6.2.1.2 Stresses in the Deck	146
6.2.1.3 Ultimate Capacity of the Bridge	146
6.2.1.4 Computation and Checking of the Deflection of the Cantilevers	147

VIII

TABLE OF CONTENTS CONT'D

	<u>Page</u>
6.2.1.5 Evaluation of Stresses Due to the Closure Operation	152
6.2.2 Design for the After the Construction Period	155
6.2.2.1 Computation of the Creep Effect on the Section Forces	155
6.2.2.2 Computation of the Secondary Moments Induced by the Post-Tensioning	155
6.2.3 Design Not Influenced by the Construction Method	157
6.2.3.1 Design of the Deck in the Transverse Direction	157
6.2.3.2 Diaphragms	160
6.2.3.3 Shear Forces and Torsion	160
6.2.3.4 Effect of Haunched Gir- ders and of Prestressing Steel on the Shear Force	163
7. EXAMPLES OF BRIDGES ERECTED BY THE CANTILEVER METHOD	166
7.1 Bendorf Bridge Over the Rhine (Federal Republic of Germany)	166
7.2 Medway Bridge (Great Britain)	169
7.3 Knight Street Bridge in Vancouver, B.C.	173
7.4 Oléron Viaduct, France	175
7.5 Grand'Mère Bridge, Province of Quebec (Under Construction)	182
7.6 Elevated Highway "Mancunian Way" in Manchester, Great Britain	186

TABLE OF CONTENTS CONT'D

	<u>Page</u>
8. CONCLUSIONS	190
REFERENCES	193
APPENDIX A	204
APPENDIX B	207

LIST OF FIGURES

	<u>Page</u>
CHAPTER 1	
1.1 Grand'Mère Bridge, Quebec, Canada	3
CHAPTER 2	
2.1 Stepping stones at Ambleside, England	5
2.2 Gladesville Bridge, Sydney, Australia	5
2.3 Early examples of concrete bridge, Glenfinnan Viaduct, Scotland	6
2.4 Maillart's Bridge over the Salgina, Switzerland, (1929)	7
2.5 Brackfriars Bridge, London, (1769)	9
2.6 Early examples of cantilever construction in Egypt, India and China	9
2.7 Proposed bridge across the Hudson River by Thomas Pope in 1811	11
2.8 The Esbly Bridge undergoing test loading	11
2.9 The units were hoisted into position in three sections	13
2.10 The three sections of the frame completed	13
CHAPTER 3	
3.1 Drau Bridge at Hollenburg, Austria, (1973)	16
3.2 Cosimato Bridge, Italy	16
3.3 Pyle Bridge, France. Construction sequences (top), casting of the extremity of the end span on scaffolding	17
3.4 Construction sequences of Tatarna Bridge in Greece	22

LIST OF FIGURES CONT'D

	<u>Page</u>
3.5 Kleinpolderplein Overpass, Rotterdam, Holland	24
3.6 Meules Bridge over the Seine near Rouen, France	26
CHAPTER 4	
4.1 Bridges with one hinge in each span	29
4.2 The Rio Pelotas Bridge in Brazil with a span of 575 feet	29
4.3 Bridges with suspended spans	29
4.4 Dead load moments for schemes 4.1.1.1 and 4.1.1.2	32
4.5 Medway Bridge, England. Erection of the suspended span	34
4.6 Continuous prestressed concrete bridges built by the cantilever method	34
4.7 Grand'Mère Bridge. Special concrete blocks (I, II, III, IV) for lapping and anchoring the prestressing steel of the bottom slab	36
4.8 Chillon Viaduct, Switzerland	37
4.9 Temporary use of stays	40
4.10 Hoëchst Bridge over the Main, West Germany. Permanent use of stays	41
4.11 Evolution of segment cross section and weights	42
4.12 Grand'Mère Bridge. Arrangement of the top slab prestressing steel	44
4.13 Parallel layout of the prestressing steel	46
4.14 "Fishbone" layout	46

LIST OF FIGURES CONT'D

	<u>Page</u>
4.15. Grand'Mère Bridge. Prestressed stirrups in the webs	48
4.16a Grand'Mère Bridge. Third group of post- tensioned steel placed in the webs	49
4.16b Grand'Mère Bridge. Third group of post- tensioned steel placed in the bottom slab	50
4.17 Choisy-le-Roi Bridge, France	51
4.18 Grand'Mère Bridge. Transverse post-tensioning of the top slab	53
CHAPTER 5	
5.1 Mobile falsework for cantilever construction	58
5.2 Schematic view of a form-traveller with concrete block and hold-down anchors	58
5.3 Pyle Bridge, France	60
5.4 Casting the segments near the pier	60
5.5 Bosques de Reforma Viaduct, Mexico. Construction sequences of the first segments	62
5.6 Grand'Mère Bridge, Province of Quebec, Canada	63
5.7 Iznajar Bridge, Spain	64
5.8 Ringbrücke over the Danube at Ulm, West Germany	64
5.9 Ounasjoki Bridge on the Arctic Circle, Finland	66
5.10 Caceres Bridge, Spain	66
5.11 Auxiliary steel girder of the Siegtal Bridge	68
5.12 Light auxiliary bridge for cantilever sections of 25 to 40 feet	68
5.13 Placing of the fresh concrete in the webs	71

LIST OF FIGURES CONT'D

	<u>Page</u>
5.14 Grand'Mère Bridge. Stressing sequences of a segment	72
5.15 Precasting of elements on a template form	78
5.16 Oleron Viaduct, France. Pier element	78
5.17 Construction sequences of the long-line method	80
5.18 Oleron Viaduct, France. Transportation of the segments	81
5.19 Schematic view of a prefabrication yard	82
5.20 Scheme of a precasting cell	85
5.21 Formwork for the short-line method	85
5.22 Turning a segment	87
5.23 Castejon Bridge over the Rio Erbo, Spain (1968)	87
5.24 Bear River Bridge, Nova Scotia, Canada (1972)	89
5.25 Saint-André-de-Cubzac Bridge over the Dordogne River, France (1974)	89
5.26 Placing the pier elements by a special structure	91
5.27 Placing the segments at the Tours-sur-Loire Viaduct	91
5.28 Bridge between the Isle of Oleron and the Continent, France (1965)	92
5.29 Cantilever support	94
5.30 Temporary cantilever prestress	94
5.31 Alpine Motorway over bridges. Erection scheme	95
5.32 Keys of a prefabricated segment	98
5.33 Forces acting upon a match-cast joint	98
5.34 Precast segment with multiple keys and internal stiffener	99

LIST OF FIGURES CONT'D

	<u>Page</u>
5.35 Ranzan Bridge in Japan	103
5.36 Moulin à Poudre Bridge, France	103
5.37 Tatarna Bridge in Greece	104
5.38 Lacroix-Falgarde Bridge, France. Temporary anchoring of the deck to the pier	104
5.39 Prefabrication of the segments and pouring phases of the Val Restel Viaduct, Italy	108
5.40 The steel nose of the bridge over the Caroui in Venezuela	108
5.41 Construction sequences of a bridge built by the pushing method	110
5.42 Two methods of placing the longitudinal prestressing steel	113
5.43 Construction phases of the Luc Viaduct, France	113
5.44 Old principle of moving the deck	115
5.45 New sliding apparatus	115
5.46 Val Restel Viaduct near Rovereto, Italy	117
5.47 Val Restel Viaduct, Italy	120
5.48 Wabash River Bridge, Covington, Indiana	121
CHAPTER 6	
6.1 Cantilever construction showing choice of span lengths and location of expansion joints	124
6.2 End span anchor in abutment for uplift	124
6.3 Grand'Mère Bridge. Use of ballast filled end spans	125
6.4 Pier stressed by bending moments due to the non-simultaneous erection of the cantilevers	127

LIST OF FIGURES CONT'D

	<u>Page</u>
6.5 Viaduct over the Lerone stream, Genova-Savona Motorway, Italy	129
6.6 Pine Valley Creek Bridge, California, USA	131
6.7 Prestressed concrete bridges erected by the cantilever method.	133
6.8 Upward forces in the bottom slab	135
6.9a Span length of the top slab according to the French Code	139
6.9b Grand'Mère Bridge cross section	139
6.10 Grand'Mère Bridge. Construction stages	142
6.11 Computer output for construction stages 101b (or 201b) of Grand'Mère Bridge	144
6.12 Computer output for construction stages 101d (or 201d) of Grand'Mère Bridge	145
6.13a Stress-strain diagram for the U.S.D. according to the F.I.P.	148
6.13b Graph for the U.S.D. according to the F.I.P. Code	149
6.14 Grand'Mère Bridge. Deflection values of the cantilevers during construction	150
6.15 Closure operations	154
6.16 Methods to introduce positive moments in the structure	156
6.17 Influence surface for the center of a plate-strip with two supported edges	159
6.18 Diaphragm design using the truss analogy method	161
6.19 Tensile forces in the diaphragms due to the bottom slab	162
6.20 Positive effect of haunched girders in shear force	164

LIST OF FIGURES CONT'D

	<u>Page</u>
6.21 Negative effect of haunched girders on shear force	164
6.22 Modification of shear force due to the prestressing steel	165
CHAPTER 7	
7.1 Bendorf Bridge over the Rhine, Germany	167
7.2 Medway Bridge, England	170
7.3 Construction phases of Medway Bridge	171
7.4 Knight Street Bridge, Vancouver. General view and construction phases	174
7.5 General view of Oleron Viaduct	176
7.6 Cross section of the viaduct	176
7.7 General view of the launching gantry	178
7.8 Launching gantry	179
7.9 Grand'Mère Bridge	183
7.10 Cross section	183
7.11 Grand'Mère Bridge form-traveller	185
7.12 Mancunian Way	187
7.13 Construction phases of Mancunian Way	188
APPENDIX A	
A1 Bridge profile and bending moments at different times for cantilever erected spans	205
APPENDIX B	
B1 Ultimate Capacity of a box girder versus an open section	208

To

My Father and Colleague

Stratis VERGANELAKIS P.Eng.

respectfully

CHAPTER I
INTRODUCTION

Of all the structures that an engineer will have the opportunity to conceive and design, probably buildings and bridges will be seen, admired or criticized the most.

But while buildings will be covered with a lot of things hiding the structural forms and dimensions, bridges will stay "naked" to show forever the revolutionary or the old-fashioned forms, the harmony or the discordance in the lines, the audacity or the conservatism of the engineer. In bridges, he will be called to create new structures, to prove his genius and go beyond the established limits and give new forms and solutions to be admired by the generations to come as the bridges of Maillart.

But for the engineer dealing with concrete structures, this fascinating field of creation will be satisfied only by the cantilever construction method, for he could exceed the accepted bounds of concrete bridges, except perhaps for concrete arches, and challenge those reserved not long ago to the steel bridges.

The object of these pages is to give to engineers a general idea of this marvellous technique of segmental cantilever construction. The material was collected at first during the prestressed concrete studies of the writer in Paris on a bursary from the French Government and later during his research for the conception and the analysis of Grand'Mère Bridge which is the bridge with the longest span (595 feet, 181 m) ever built with this system in North America (1977).

Anyone who may be interested in knowing more on this subject could find many articles and books treating this subject in theory and in practice, some of which are given in the references at the end. Especially, the reference 1.1, published by the Portland Cement Association, gives a vast bibliography (117 references) with abstracts of segment post-tensioned construction between the years 1962 and 1967.

PREVIOUSLY COPYRIGHTED MATERIAL,
GRAPH ON LEAF 3,
NOT MICROFILMED.

COPYRIGHT: GOUVERNEMENT DU QUEBEC
MINISTÈRE DES TRANSPORTS
DIRECTION GÉNÉRALE DU GÉNIE

GRAPH OF GRAND'MÈRE BRIDGE

CHAPTER 2

HISTORY

From the first stepping stones put across the shallow parts of a river (Fig. 2.1, Ref. 2.1) to the modern bridges of today (Fig. 2.2, Ref. 2.1) the evolution was tremendous and their history a fascinating subject.

The brief historical outline given hereafter will be limited to the concrete bridges only. The reader who will like to know more about this evolution will find a lot of books dealing with this subject (Ref. 2.1).

The construction of concrete bridges started during the latter half of the last century, but since the use of reinforcement was unknown at that time, these structures were a replica of the arches of the past (Fig. 2.3, Ref. 2.1).

At the turn of the century (Ref. 2.1) engineers (as Hennebique and others) introduced steel bars in the concrete in order to resist the tensile stresses. The use of this new material, the reinforced concrete, permitted Maillart to build his famous bridges (Fig. 2.4, Ref. 2.1).

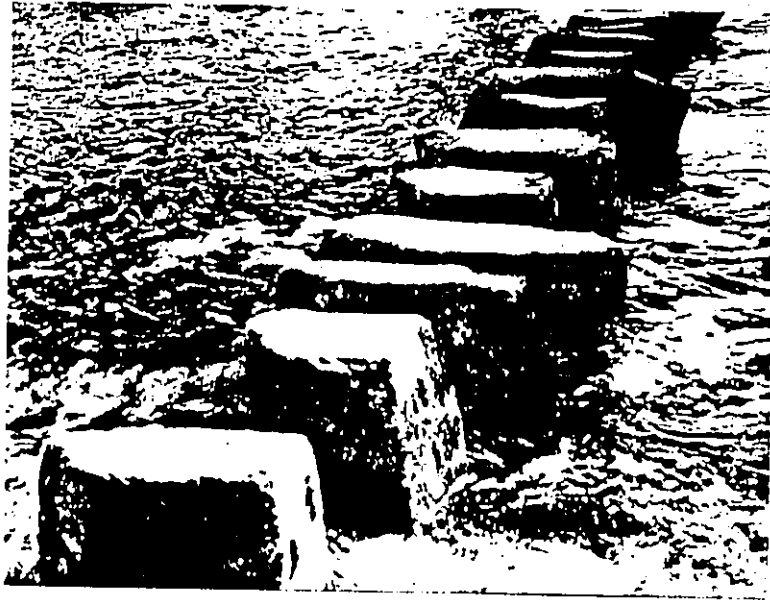


Fig. 2.1 Stepping stones at Ambleside, England

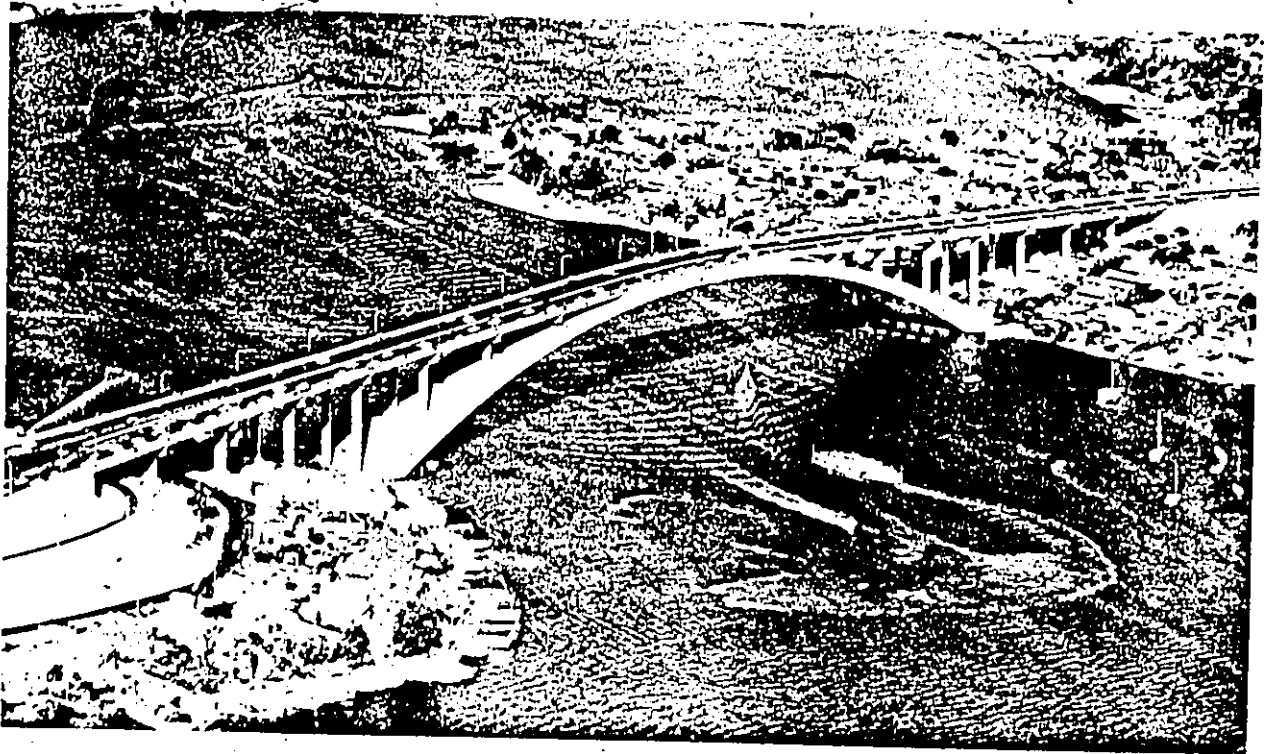


Fig. 2.2 Gladesville Bridge, Sydney, Australia

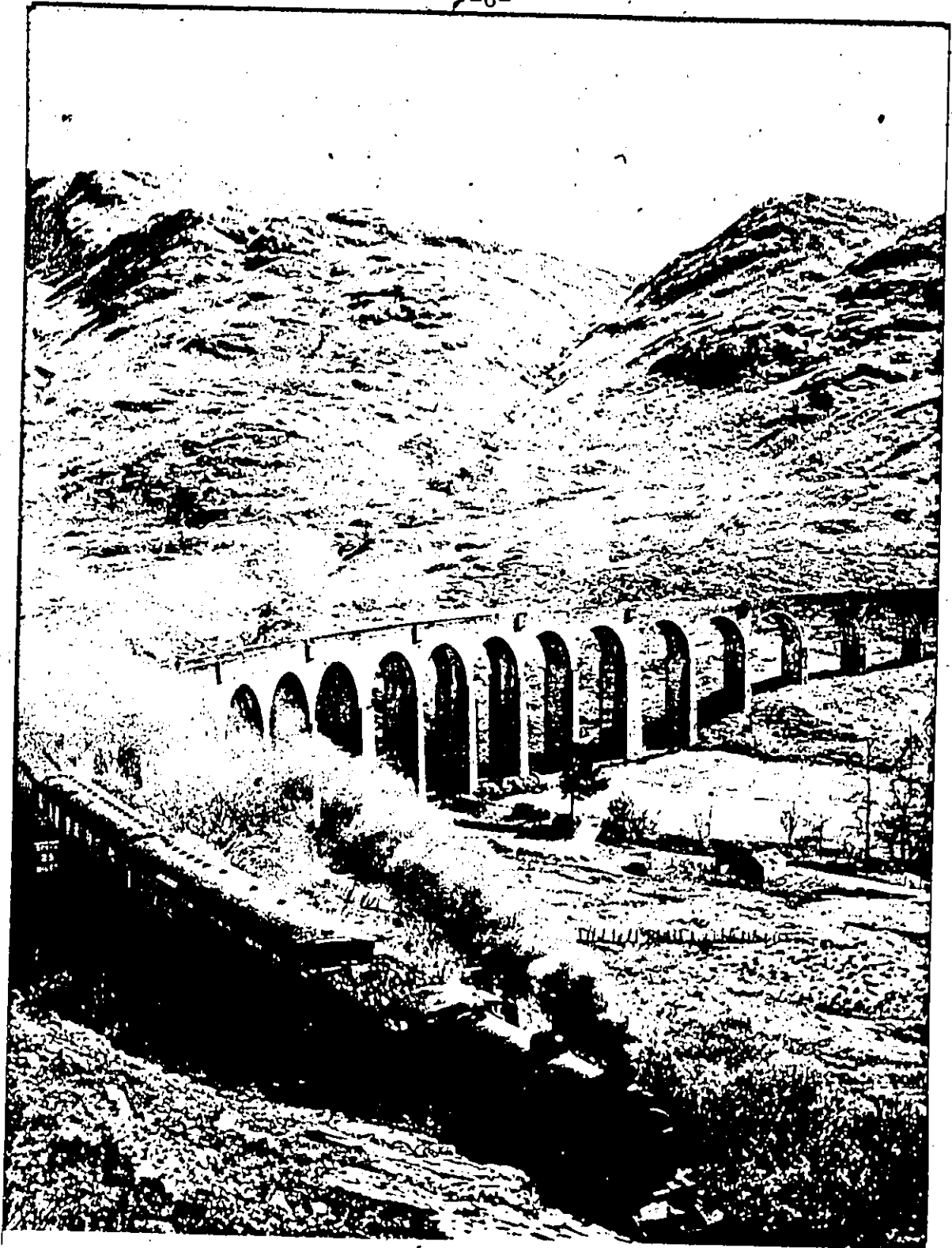


Fig. 2.3 Early example of concrete bridge, Glenfinnan Viaduct, Scotland.

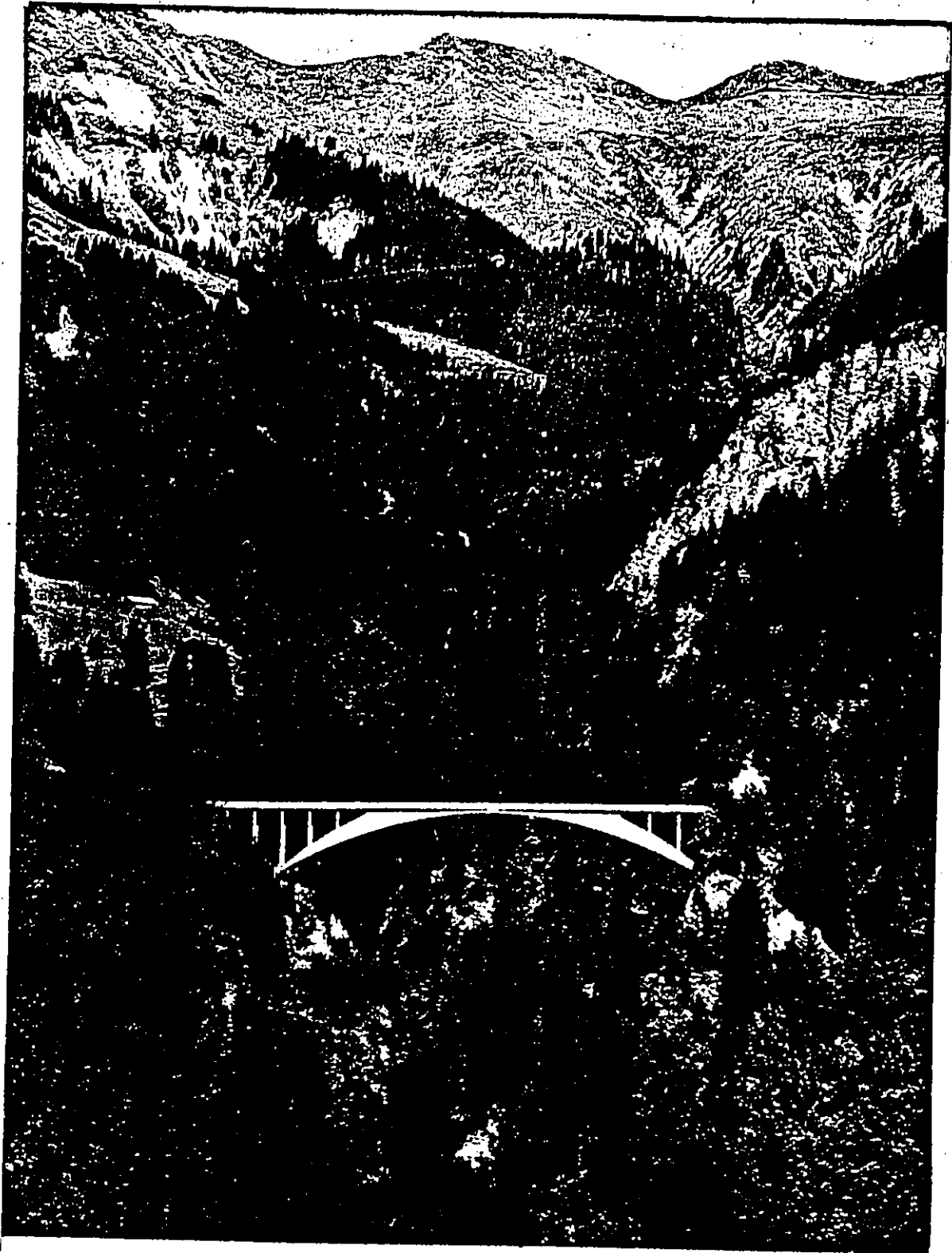


Fig. 2.4 Maillart's Bridge over the Salgina, Switzerland, (1929).

The greatest progress in the field was realized during the twentieth century with the coming of prestressed concrete, developed chiefly by the great Eugène Freyssinet, which eliminates the tensile stresses and cracks by applying an initial compression to the structure. But it was only in the early fifties, with the development of the cantilever erection method, that new horizons were given to the concrete bridges.

To build a bridge in reinforced concrete, as the stone arches in the older days (Fig. 2.5, Ref. 2.1), a scaffolding must be erected which, although a temporary structure, must have sufficient capacity to carry the construction loads and so increase the total cost of the bridge. In addition, if the bridge crosses a river with large flood plains or a deep valley, the price of these temporary structures can be excessive compared to the total cost of the finished work.

Alternatively, if it is possible to erect the bridge in small segments cantilevered from the piers or the abutments and anchored each time on the finished and self-supported part of the deck, then this problem is solved.

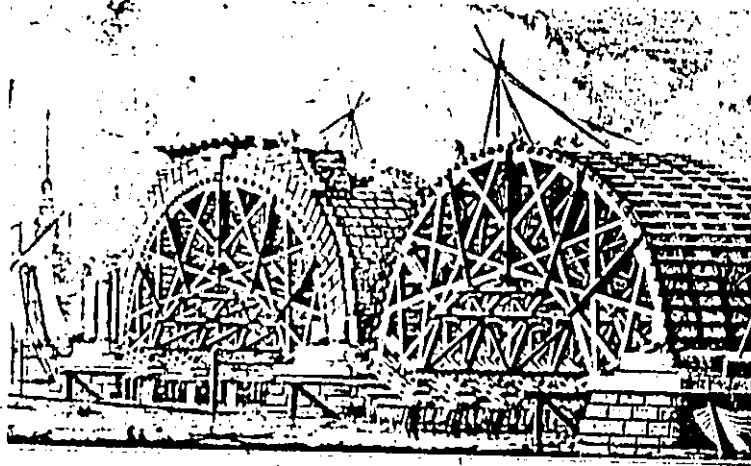


Fig. 2.5 Brackfriars Bridge; London, 1769.

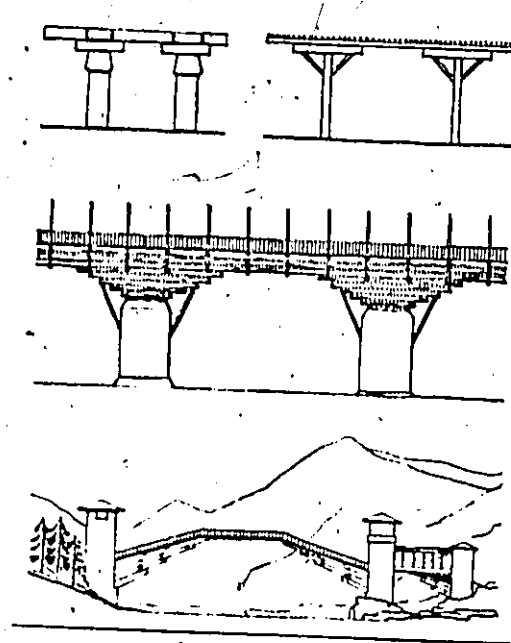


Fig 2.6 Early examples of cantilever construction in Egypt (top), India and China.

The cantilevering construction method, a very simple idea indeed, is not a new approach to building. In the ancient times (Ref. 2.1) similar methods were used in Egypt, India and China (Fig. 2.6, Ref. 2.1). In Egypt, it was the capital-lintel system, in India, a piling of wood beams, each piece of different length, was used to form the deck, and finally, in China, wood beams corbelled from masonry masses made the structure.

The first application, in the modern times, of the cantilevering method was used for steel bridges (Ref. 2.1). The first long span bridge built by this method was the Forth Bridge, in 1890, with a main span of 1700 feet (518 m). Although an American engineer, Thomas Pope, proposed in 1811 the construction over the Hudson River of a bridge with a main span of 1800 feet (549 m) erected with prefabricated wood trusses corbelled out of the abutments (Ref. 2.2).

Later (Ref. 2.1), in 1948, Eugène Freyssinet used a similar method for the five bridges over the Marne River near Paris (Fig. 2.8). Each bridge, with a main span of 240 feet (80 m), was composed of prefabricated beams cut in five segments. Once the two exterior pieces were erected



Fig. 2.7 Proposed Bridge across the Hudson River
by Thomas Pope in 1811.

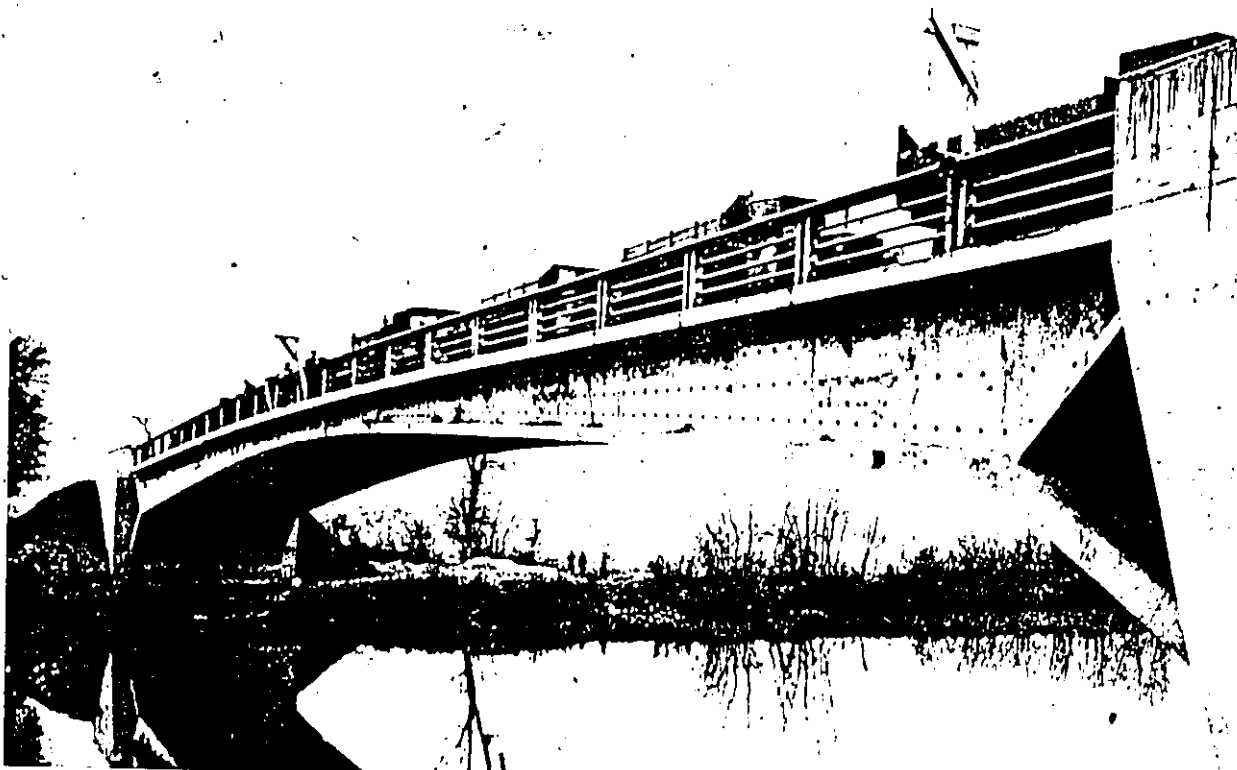


Fig. 2.8 The Esbly Bridge undergoing test loading.

and anchored to the abutments, the three other segments were placed using cranes and guys (Fig. 2.9). When all five segments were positioned and their joints filled with concrete (fig. 2.10) a prestressing force was applied to consolidate and complete the structure.

But the first real application of this method to concrete structures, as we know it today, appeared only in the early fifties with the reconstruction of the Worm's Bridge, called Niebelungen Brücke, over the Rhine and the other one over the Moselle near Coblenz by the German company Dyckerhoff & Widmann.

Here is, in a free translation, how an engineer described the impression made by these bridges at that time (Ref. 2.3).

"After the destruction of all the bridges over the Rhine and on the majority of its affluents on the left bank, many structures were completed during the past few years, some being the outcome of new techniques. The most note-worthy are the prestressed concrete bridges at Worm over the Rhine and at Coblenz over the Moselle.

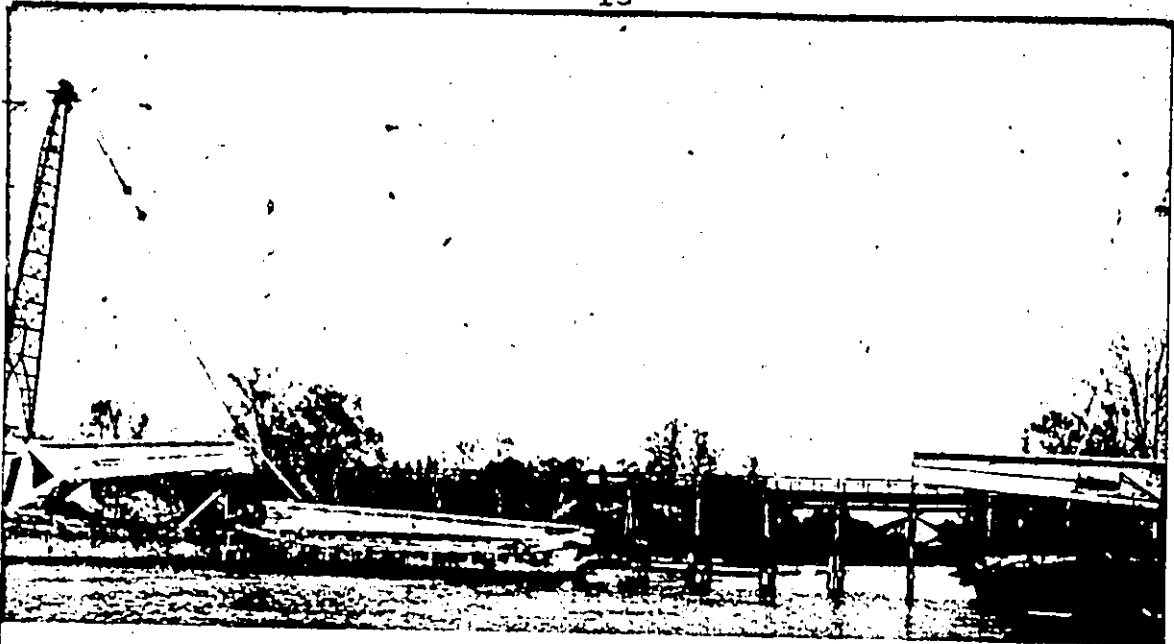


Fig. 2.9 The units were hoisted into position in three sections.

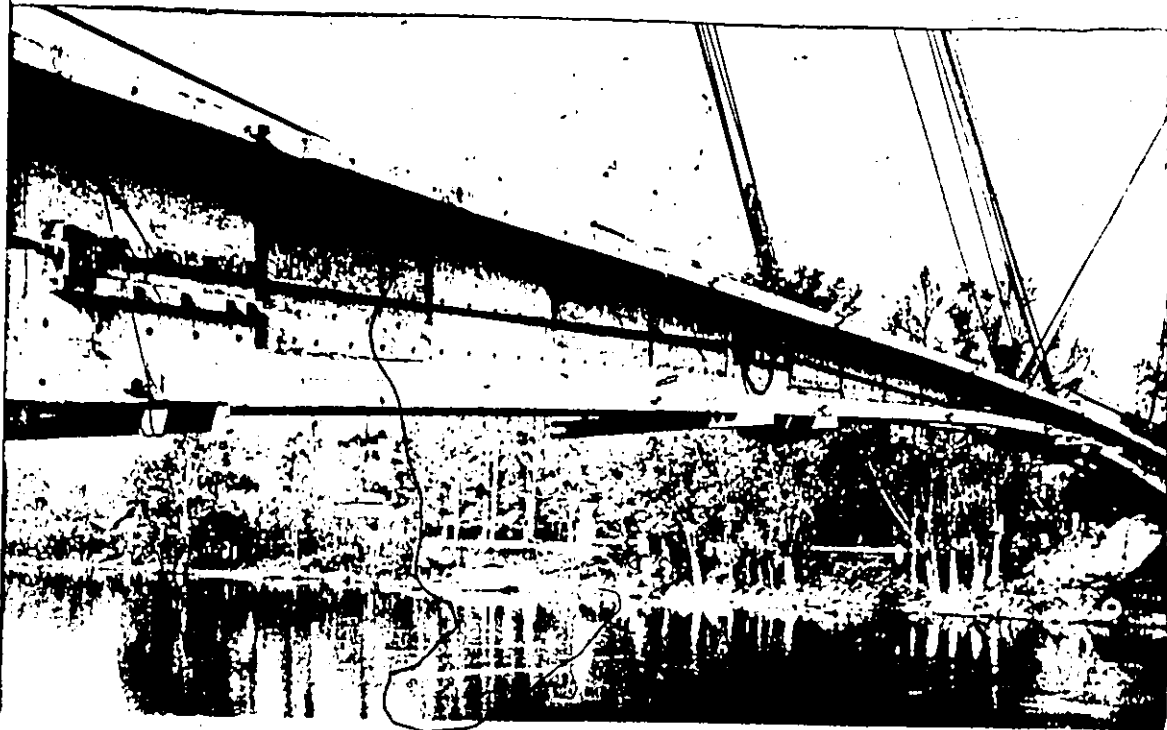


Fig. 2.10 The three sections of the frame completed.

The principal interest of these appealing structures lies not only in their spans, which although long, are not records, but mainly for the new construction methods adopted.

Beyond an ingenious new system of stressing the bars, the construction method is in some way the application to reinforced concrete the erection methods used for steel structures, and opens unsuspected horizons to prestressed concrete....".

And he concludes:

"The bridges at Coblenz and Worm, the latter one being the first in prestressed concrete, over the Rhine are of impeccable aesthetics. They do honour to their constructors, the Dyckerhoff & Widmann company, and open wide future prospects especially because of the use of new methods of stressing the bars and the cantilever erection".

Since then, the prophecy of this engineer was proven true. With the cantilever method adopted throughout the world, the prestressed concrete bridges have achieved an extraordinary success reaching spans that nobody could imagine just twenty years ago.

CHAPTER 3

DESCRIPTION

3.1 Building with Blocks

The cantilever construction method consists of erecting the bridge deck, starting on top of a pier, by small segments forming a system (called hammerhead) of two symmetrical cantilevers, one on each side of the pier. (Fig. 3.1, Ref. 3.1).

Each segment is cast, not in a formwork on top of a conventional scaffold, but in a special structure (called form-traveller) anchored on the finished part of the deck (Fig. 3.2, Ref. 3.2).

Once the required concrete compression strength is obtained, the segments are consolidated one to the other by prestressing them and thus forming two self-supported cantilevers. This cycle is repeated until one half-span on each side of the pier is completed (Fig. 3.3, Ref. 3.3)

This process is repeated on each pier in order to complete the whole bridge, including the exterior spans whose length is normally equal to the first pier cantilever.

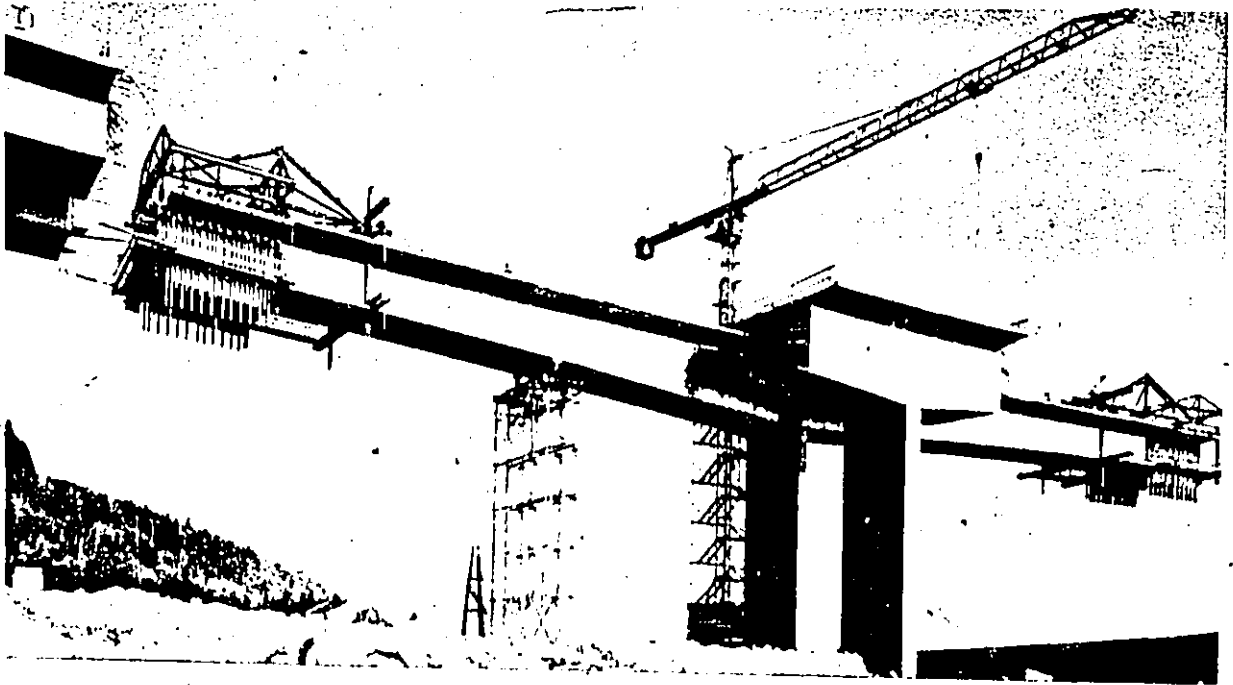


Fig..3.I Drau Bridge at Hollenburg,Austria,1973.

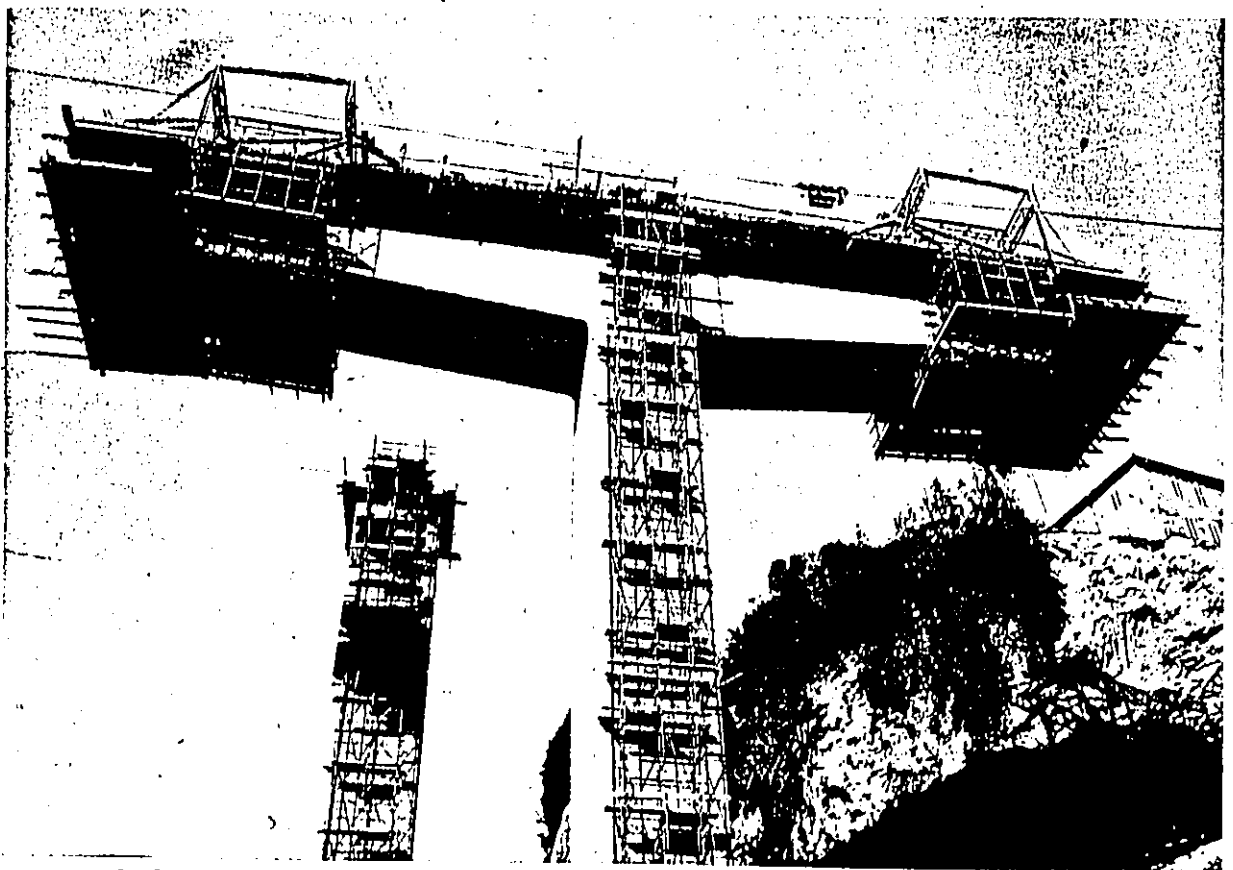


Fig. 3.2 Cosimato Bridge,Italy.

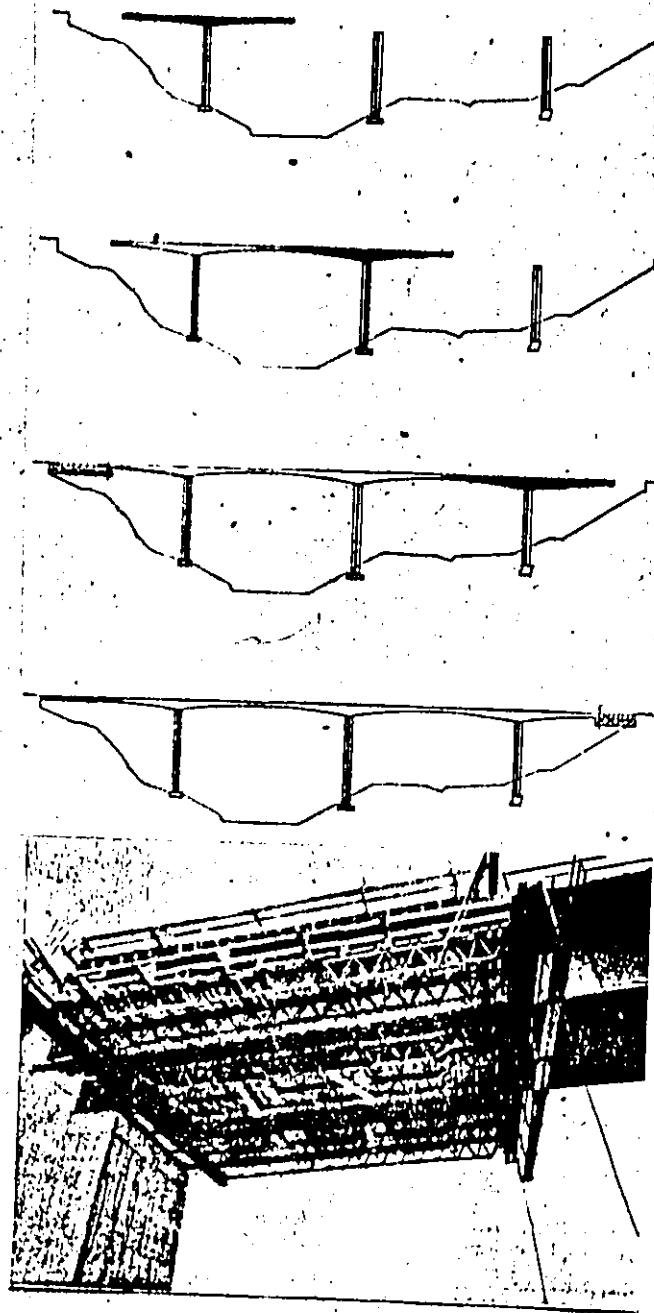


Fig. 3.3 Pyle Bridge, France. Construction sequences (top), casting of the extremity of the end span on scaffolding.

If the exterior span is of a different length, the remaining part is cast in place on a scaffolding or built by cantilevering it from the abutment.

At the center of each span, a closure pour connects the adjacent cantilevers to provide continuity. A hinge or a suspended girder can also be used to close the bridge.

It is clear that building a bridge with small blocks eliminates the scaffolding and so facilitates the construction over wide rivers or ones with big flood plains or bridges with very high piers, and reduces to a minimum the formwork, since the one used to cast the first segment will be used to make all the others.

3.2 Advantages

/ To better appreciate the cantilever method one has to look upon the disadvantages of the traditional cast-in-place method.

- (a) Most bridge collapses occur during construction not because of an incorrect deck design, for this was the object of a serious and elaborate analysis, but due to the structural insufficiency of the scaffolding. Because of the scaffolding's temporary use and of its sometimes high cost, a minor importance is given, many details may be omitted, smaller safety factors may be used or an excessive soil capacity loading under them is considered.
- (b) The scaffolding, as well as the formwork, is always different from one structure to the other depending on the ground profile, the shape of the deck and the weight of the cast-in-place concrete.
- (c) In cities, the cast-in-place construction may be an obstruction to the traffic and sometimes must be completely excluded.
- (d) If the piers are very tall, the cost of these temporary structures can be excessive compared to the total cost.

- (e) By using scaffoldings the work progress is more dependent on weather conditions, except if a shelter, usually expensive, covers the whole structure.
- (f) A larger number of men and more machinery are required due to the requirement to install the formwork, the steel, and the concrete over the whole deck at the same time.

Of course, many of these inconveniences can be eliminated by using prefabricated beam decks (although the top concrete slab is to be cast in place) but their use is limited to spans of between 120 and 150 feet (37 and 46 m) because of vibration problems (AASHO girders), their excessive weight (specially made beams), and transportation problems.

On the contrary, the cantilever method, in addition to eliminating all these disadvantages, offers the possibility of a better control and a finished product of superior quality. This can be achieved by concentrating the work of a small and highly trained crew, due to the mechanization of work, on a small and completely sheltered

part of the bridge at any one time.

In addition, it allows someone to start the deck construction before the substructure is completely finished, by starting for example the deck at one pier, while at the same time the other piers and the abutments are under construction (Fig. 3.4, Ref. 3.5).

It is also possible, even before starting the site work at all, to prefabricate the segments in a factory during the winter.

Therefore, with the cantilever method the deck construction does not depend on the profile or the nature of the ground, the weather conditions or the manpower available.

3.3 Field of Application

The application of the cantilever erection method starts with spans greater than 165 feet (50 m) (Ref. 3.10) because under this limit the prefabricated beams, the cast-in-place deck or the pushing method can be more competitive in total cost (Ref. 3.10).

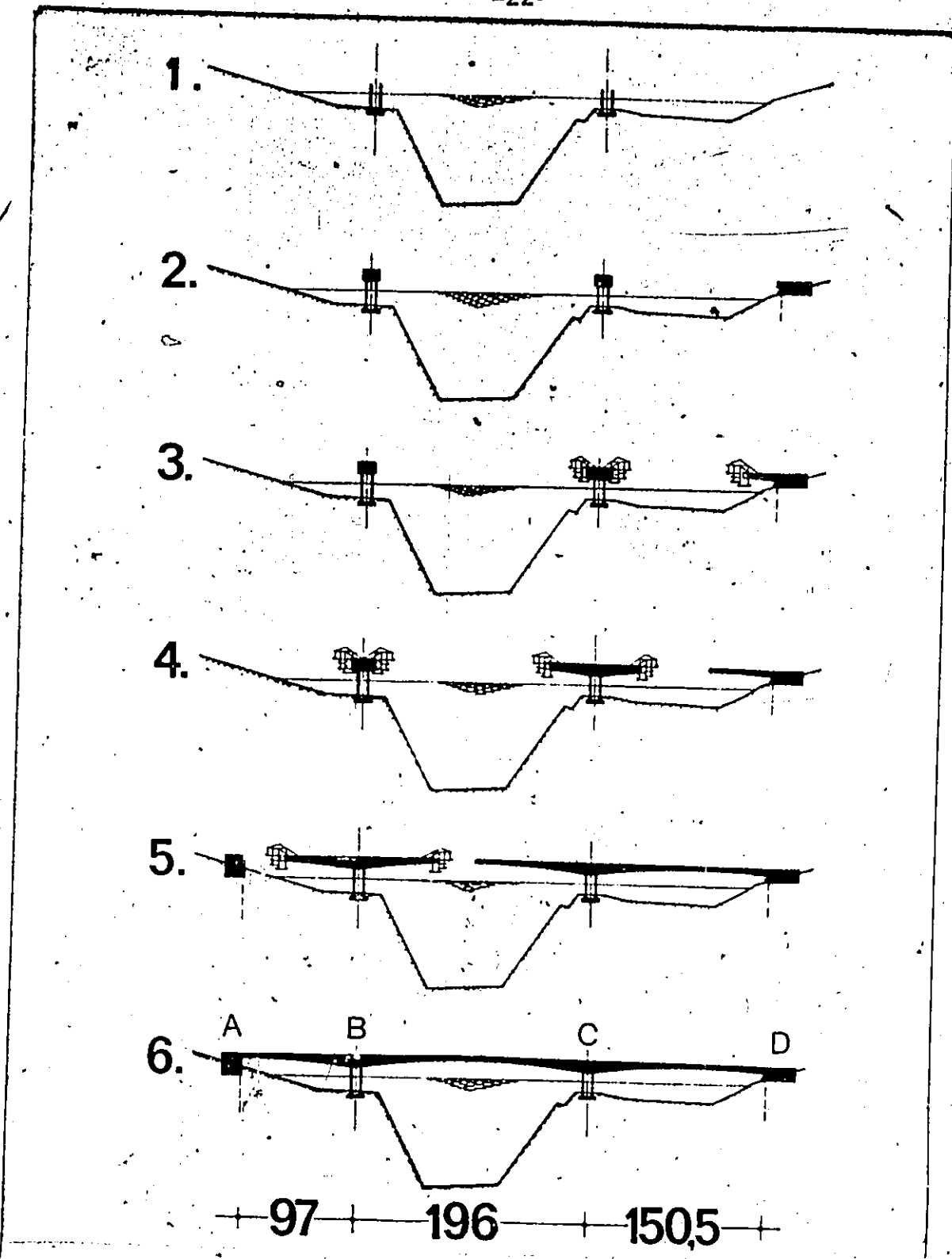


Fig. 3.4 Construction sequences of Tatarna Bridge in Greece.
(Dimensions in meters).

However, if the local conditions or the site requirements are suitable or a significant number of similar bridges are given to the same general contractor, the cantilever method can successfully compete with the other systems even on bridges with smaller spans. Such was the case, in 1971, when the Kleinpolderplein overpass near Rotterdam in Holland was completed (Ref. 3.6), formed by three viaducts of a total length of almost 1.25 miles (2,000 m) and located in an urban area with very dense traffic which could not be interrupted, these viaducts, with spans between 85 and 115 feet (26 and 35 m), were constructed by the cantilever method with prefabricated segments brought to the site on trucks and put in place by a special steel gantry without any noticeable traffic disturbance (Fig. 3.5, Ref. 3.6).

Does an upper limit exist? Every day new record spans are established and very soon (Ref. 3.7), by using the cantilever method combined with the cable-stay, the prefabrication (for the webs) as well as the cast-in-place (for the slabs), the record span 1,050 feet (320 m)

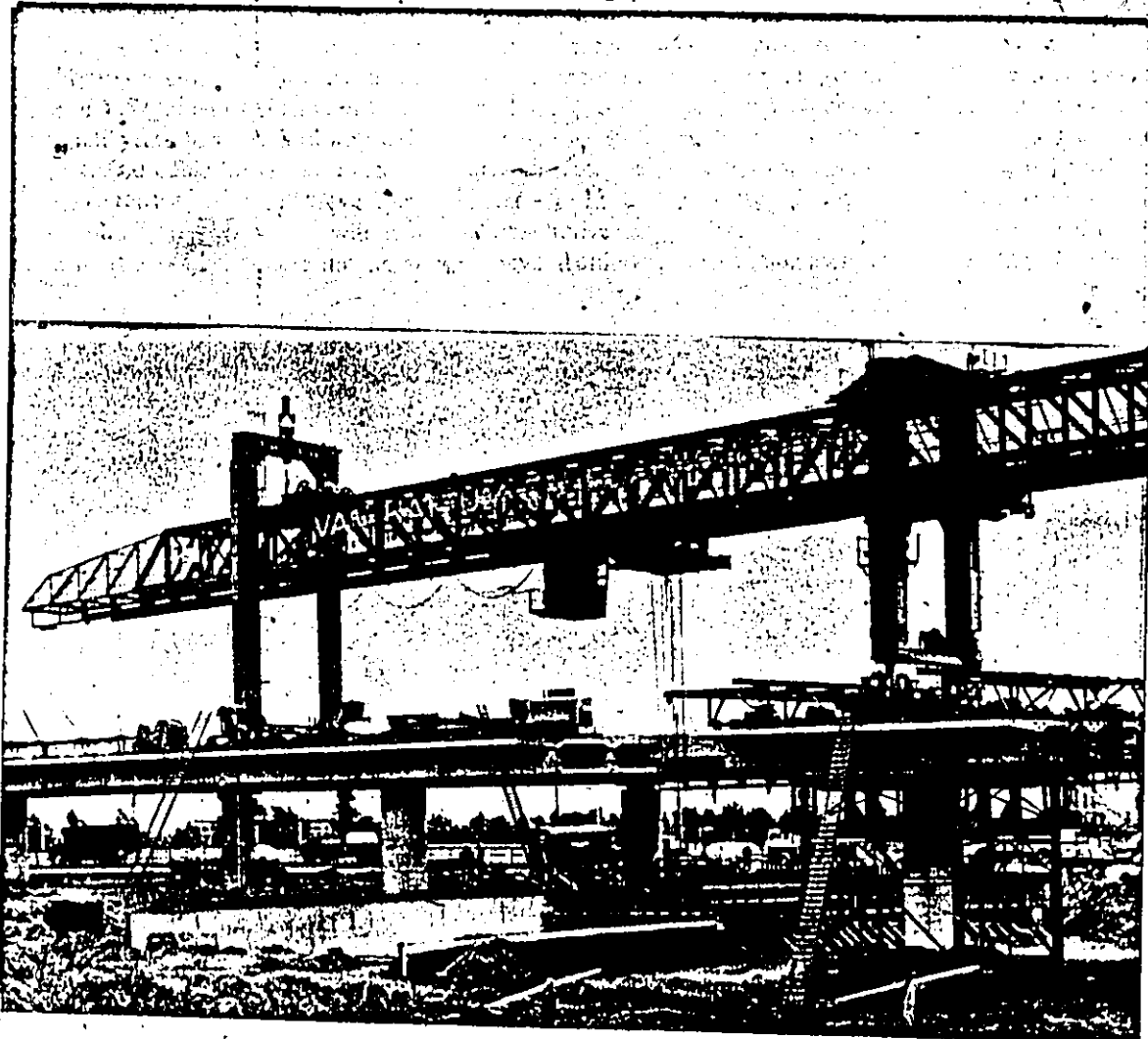


Fig. 3.5 Kleinpolderplein Overpass, Rotterdam, Holland. The steel gantry used for the erection of the prefabricated segments.

will be attained (Meules Bridge, France, Fig. 3.6). It is difficult to predict what the limit will be especially with the use of light-weight concrete.

In conclusion, in spite of the opinion of the writer that a special field of application does not really exist, depending on local conditions, the contractor's experience and equipment or the engineer's audacity, this limit, according to many writers (Ref. 3.8 and 3.9), is somewhere between 165 and 395 feet (50 and 120 m) with an optimum value between 195 and 295 feet (60 and 90 m).

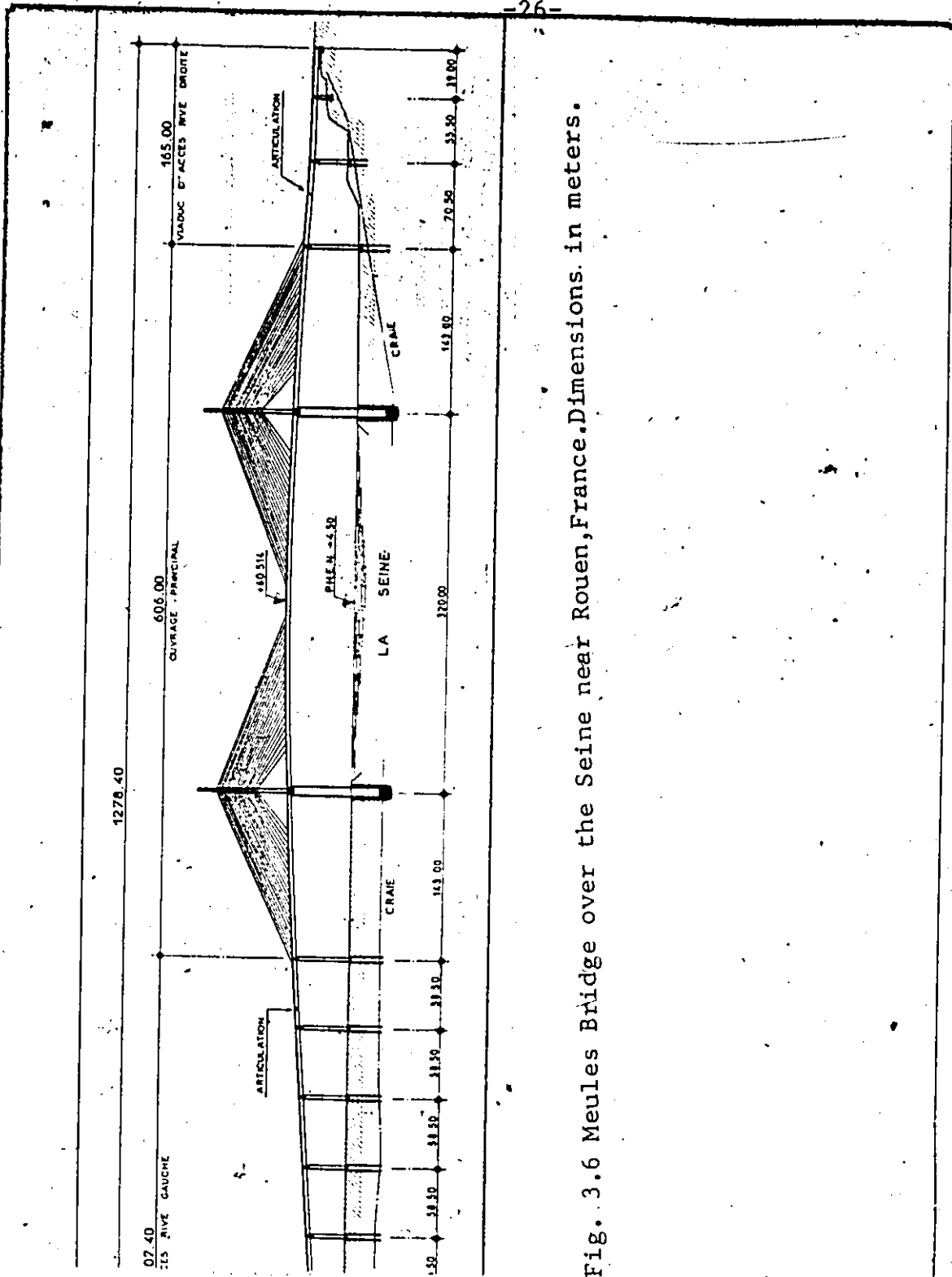


Fig. 3.6 Meules Bridge over the Seine near Rouen, France. Dimensions in meters.

CHAPTER 4

STATICAL SYSTEM AND PRESTRESSING STEEL LAYOUT

4.1 Lengthwise Statical System

Although the choice of the statical system depends on local conditions or on the shape of the bridge itself, nevertheless there was a considerable evolution between the first structures having a hinge at the center of each span and the continuous ones in general use today. This change is mostly due to a better knowledge of the materials and more powerful design instruments (computers, etc.).

Generally speaking, the statical systems used today are:

- (a) The system with one or two hinges in almost every span;
- (b) The continuous one;
- (c) The cable-stay system.

4.1.1 Bridges with One or Two Hinges in Each Span

4.1.1.1 Bridges with One Hinge (Fig. 4.1, Ref. 4.1)

The first bridges built by the cantilever method had a hinge at the center of each span capable of transferring from one cantilever to the other expansion movements and shear forces but no bending moments.

The advantage of this system lies in the design facility because the system is statically determinate for the construction dead loads and indeterminate only for the live loads or the loads applied after the bridge is completed. Even in the latter case, the redundant force is the vertical reaction in the hinge (one degree of indeterminacy).

In spite of this facility which, due to the sophisticated computer programs of today, is no longer an advantage, this system is less and less used because of its big drawback, the discontinuity in profile.

In fact, due to the creep of the concrete, the two cantilevers undergo deformation with time which results in a progressive deflection of their extremities and the formation of a break or an angular point in the lengthwise

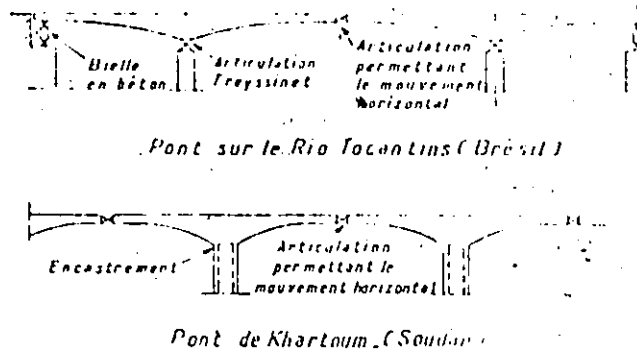


Fig. 4.1 Bridges with one hinge in each span.

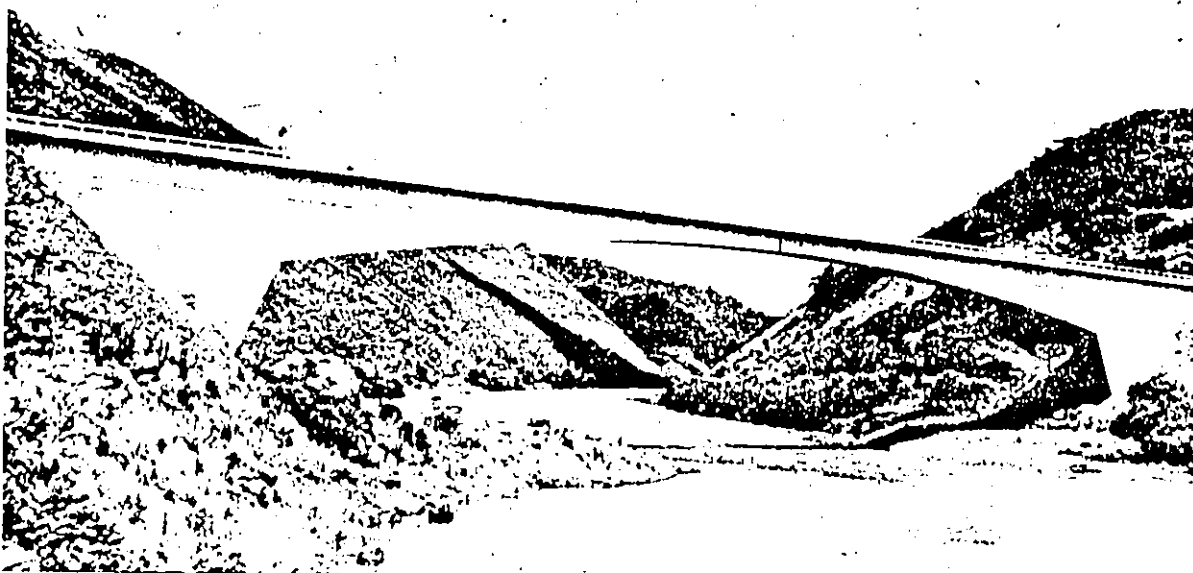


Fig. 4.2 The Rio Pelotas Bridge in Brazil with a span of 575'-0

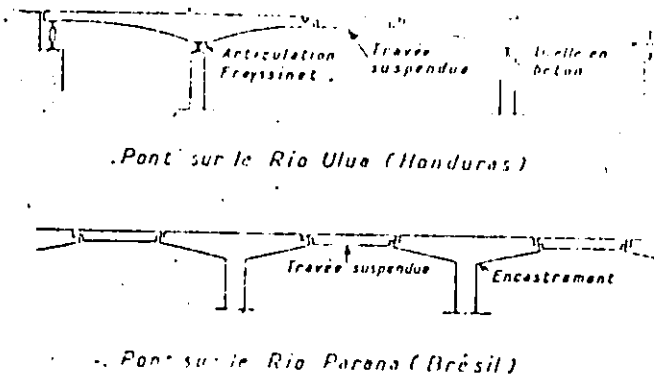


Fig. 4.3 Bridges with suspended spans

profile just over the hinge. Although the bridge capacity is not affected, the overall appearance is disagreeable and the resulting sag is a discomfort for the traffic.

However, this system is the only one used for cantilevers anchored to bedrock (Rio Pelotas Bridge in Brazil (Fig. 4.2, Ref. 4.2)) where the only possible expansion movement must be accommodated by the hinge.

4.1.1.2 Bridges with Two Hinges (Fig. 4.3, Ref. 4.1)

A substantial improvement regarding the profile break is to put two hinges in the same span by suspending a short portion of the deck from the two cantilever edges.

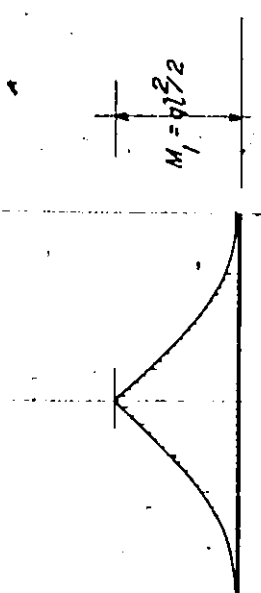
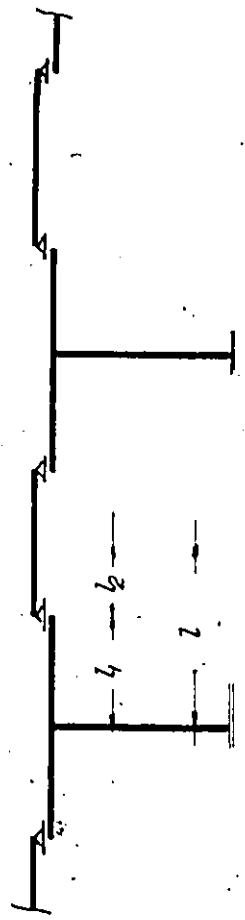
This system, in addition to a simple design, has the following advantages:

- (a) If the cantilever edges are not at the proper level due to an incorrect camber calculation or a wrong control during construction, the suspended span can easily accommodate this small level difference.

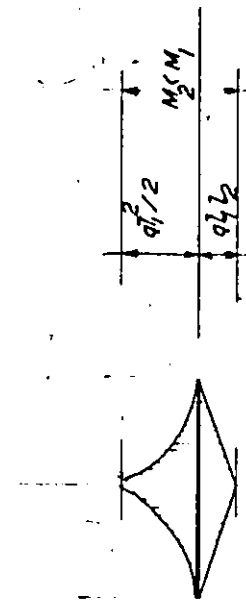
- (b) The longitudinal profile break appears less.
- (c) The moment over the pier is smaller than that found for a similar span with only one hinge for the following reasons:
 - (i) The bending moments are not negative throughout the span because they are zero at the end of the cantilever and are positive in the suspended span (Fig. 4.4).
 - (ii) As the suspended span has only positive moments, it can be composed of T type beams with a small bottom flange and a lesser weight per linear foot, compared to the box section.
- (d) This is an ideal solution in cases where foundation settlements are to be expected.

But the system presents some drawbacks such as:

- (a) An unpleasant appearance produced by the discontinuity of the deck profile between the



Scheme 4.III



Scheme 4.II.2

Fig.4.4 Dead load moments for schemes 4.I.II et 4.II.2.

variable moment of inertia of the cantilevers and the constant moment of inertia of the suspended span.

- (b) The necessity to use two different types of erection equipment, that is, a form-traveller for the corbels and a launching girder for the suspended beam (Fig. 4.5, Ref. 4.3).

Both these drawbacks were overcome by Dywidag Canada Ltd. at the Knight Street Bridge in British Columbia (see Chapter 7).

4.1.2 Continuous Deck (Fig. 4.6, Ref. 4.1)

The first continuous bridges built by the cantilever method were in France in the early sixties (Choisy-le-Roi Bridge, 1963-1964, Ref. 4.7), and since then their use has been generalized throughout the world.

Generally speaking, these structures are erected in the classical cantilever way, but once the two corbels are finished, the gap between them is closed by a cast-in-place concrete section. To achieve the continuity, post-

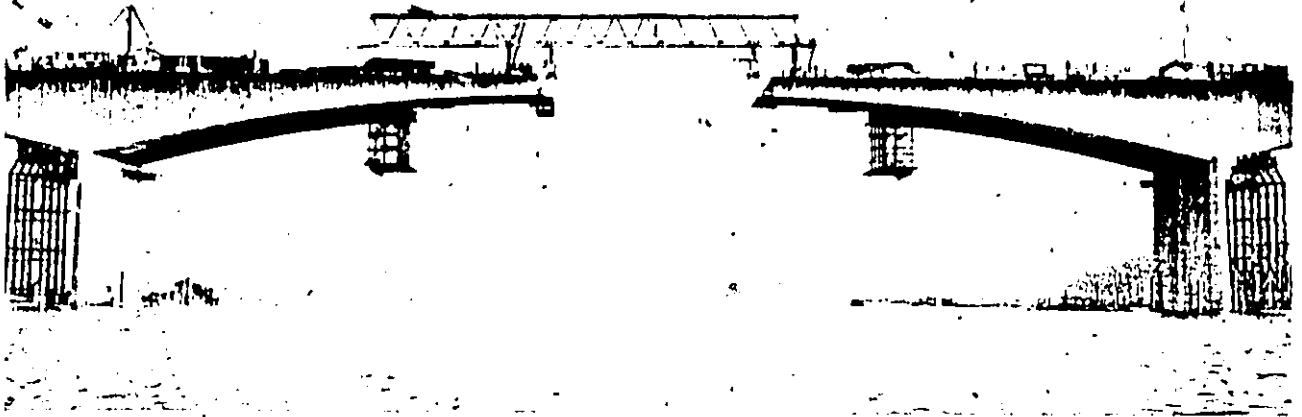


Fig. 4.5 Medway Bridge, England. Erection of the suspended span

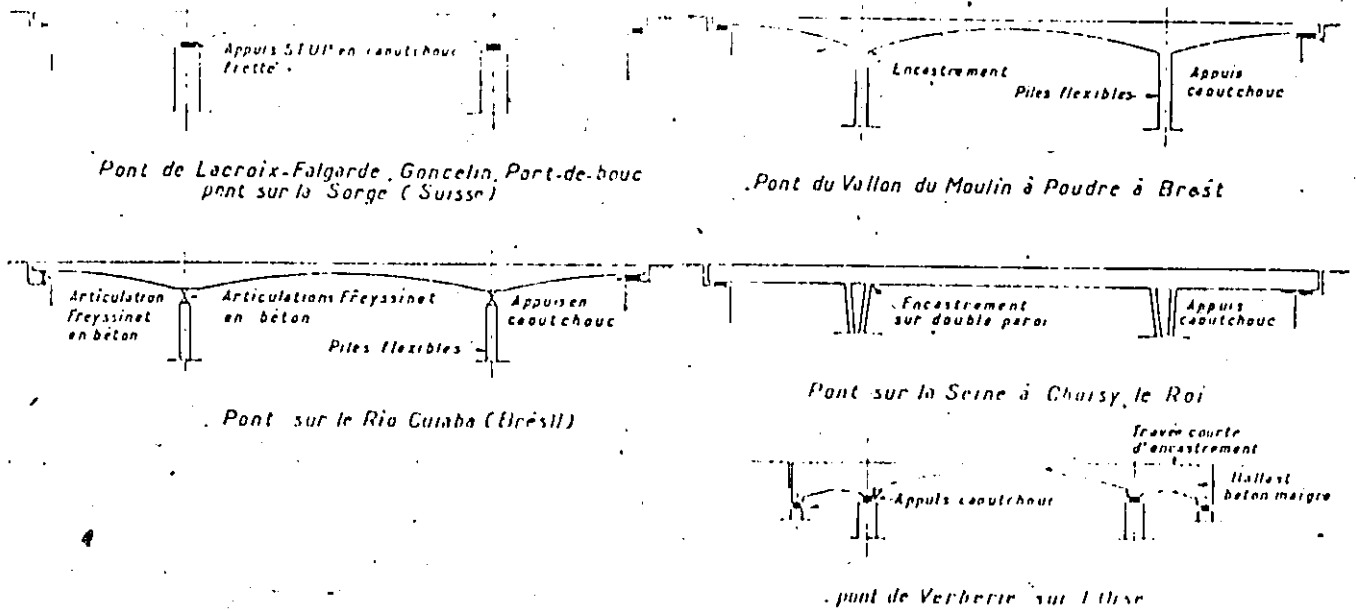


Fig 4.6 Continuous prestressed concrete bridges built by the cantilever method.

tensioning steel is placed in the region of the gap. This steel will resist the bending moments created by the dead loads, applied after the closing operation, the traffic live load and those induced by the creep of the concrete discussed hereafter. Since all these moments are positive, the prestressing steel is placed either in the bottom slab and anchored in special blocks (Fig. 4.7), or in the webs of the box section using parabolical cables anchored in the top slab.

They are graceful structures, as the Chillon Viaduct (Fig. 4.8, Ref. 4.4), without any break in their lengthwise profile and able to follow in plan and elevation the ground relief. Their design is more difficult but computers have greatly simplified this task.

A special characteristic of this kind of structure is the redistribution of the bending moments after the closing operation due to the creep of the concrete (Ref. 4.8 and 4.9). This particularity occurs in structures erected under a statical system and then modified to another type once the bridge is completed.

PREVIOUSLY COPYRIGHTED MATERIAL,
GRAPH ON LEAF 36,
NOT MICROFILMED

COPYRIGHT: GOUVERNMENT DU QUEBEC
MINISTÈRE DES TRANSPORTS,
DIRECTION GÉNÉRALE DES GENIE

GRAPH OF GRAND-MÈRE BRIDGE

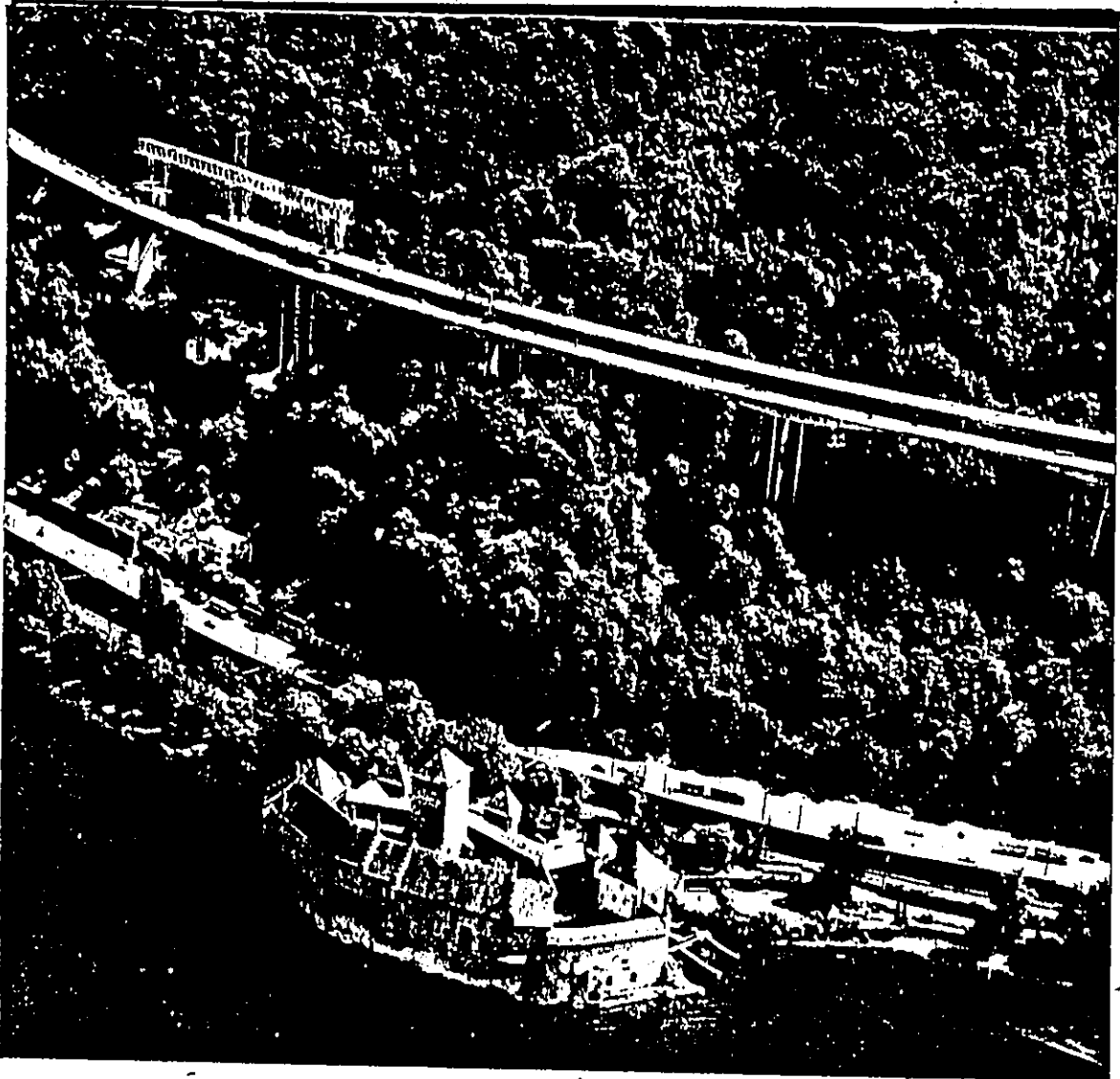


Fig. 4.8 Chillon Viaduct, Switzerland.

The total length of 6890 feet (2100.m) is divided
in 23 spans (maximum span 341 feet, 104.m)

So, the cast-in-place on a scaffolding structures will undergo, due to creep, a change of their elastic curve but with no additional stresses or changes of the section forces. Conversely, the structures with a statically determinate system during erection (cantilevers) and an indeterminate one after the construction is finished, will undergo not only a change of the elastic curve but also a modification of the section forces. So the negative moments of the cantilevers due to the dead loads will remain unchanged if no modification is made to the erection statical system, but once the cantilevers are closed a positive moment will occur in the middle of the span with a corresponding reduction of the negative moment over the pier (see Appendix A).

4.1.3 Cable-Stayed Bridges

These bridges are built in the classical manner of cantilevering, but as the construction progresses the corbels are suspended with stays from concrete or steel pylons erected over the piers.

The stays can be used either temporarily during erection in order to reduce the section loads (Fig. 4.9, Ref. 4.5), or permanently (Hoechst Bridge in Germany, Fig. 4.10). In the first case, the effect upon the stresses of the structure of removing the stays has to be investigated.

4.2 Transverse Statical System (Fig. 4.11, Ref. 4.10)

The cantilever erection method, with negative moments throughout the length of the deck during construction, requires the placement of the post-tensioning steel on the top of the section and at the same time requires a thick bottom slab to resist the compression stresses. The most suitable section under these conditions is the box. In fact, an open section with a thick bottom flange presents some serious drawbacks such as:

- (a) Difficulties with formwork.
- (b) Reduced torsional rigidity.

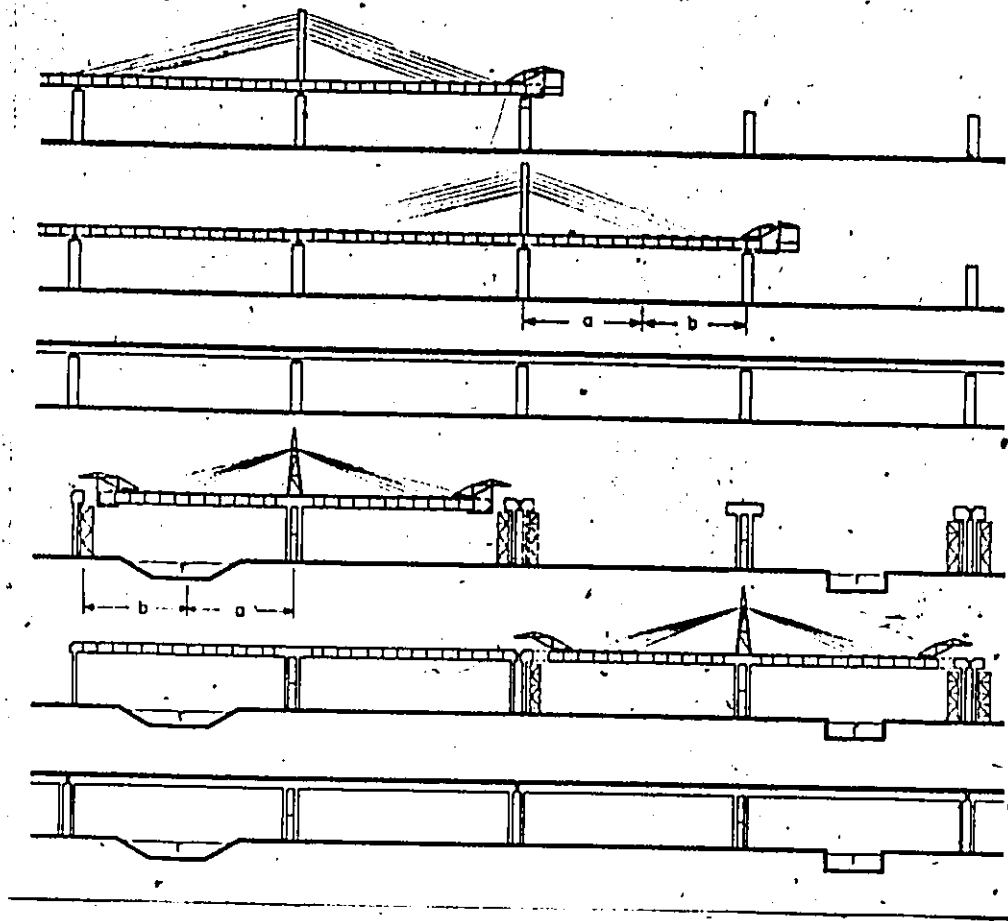


Fig. 4.9 Temporary use of stays.

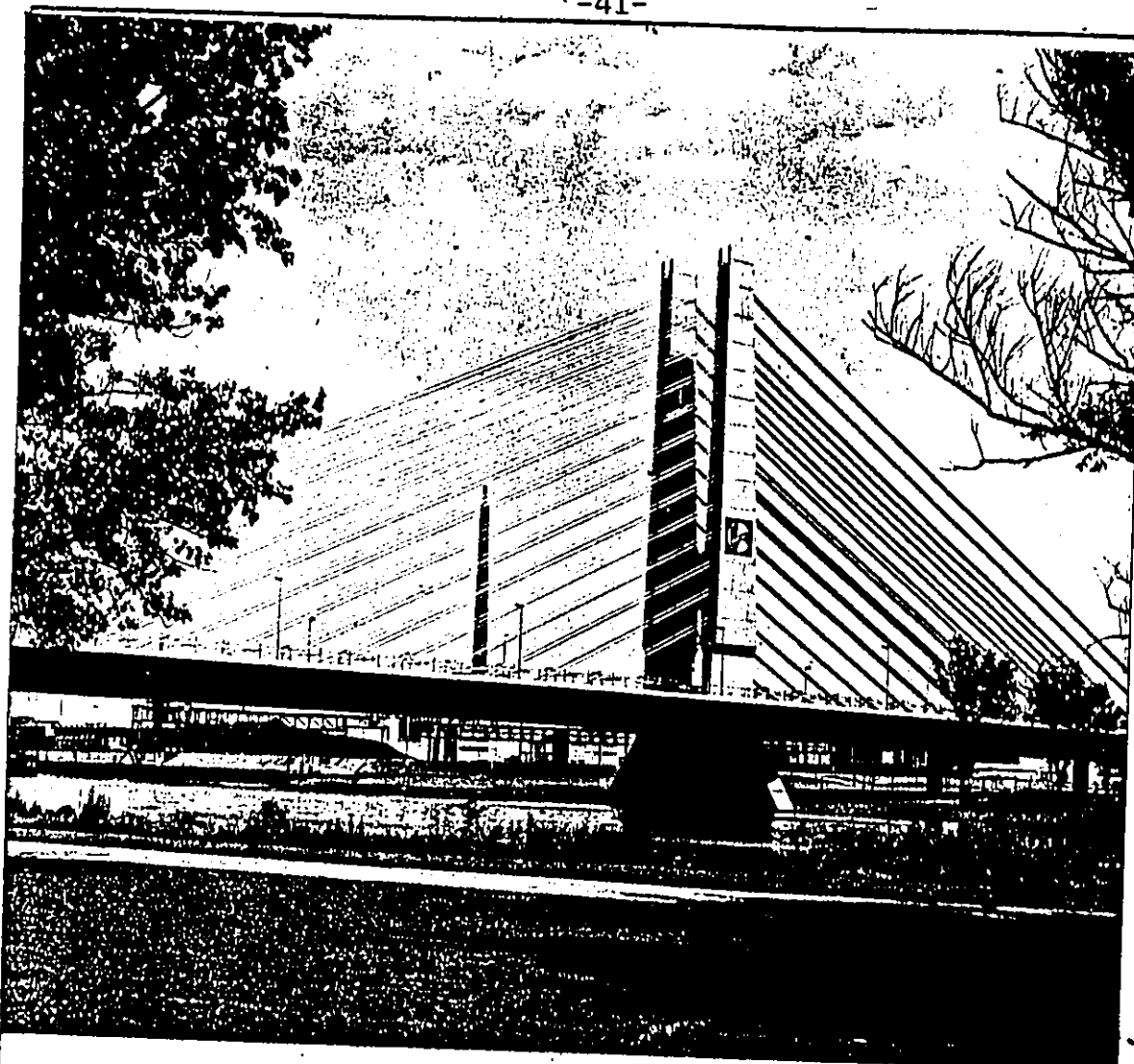


Fig. 4.10 Hoechst Bridge over the Main, West Germany.

Permanent use of stays. With a main span of 486 feet it is the longest railway bridge in prestressed concrete.

BRIDGE	CROSS-SECTION (Dimensions in meters)	-42- SEGMENT LENGTH	MAXIMUM SEGMENT WEIGHT (Tonnes)
CHOISY-LE-ROI		2.50 m 8.20 ft	25
SEUDRE		3.30 m 10.80 ft	75
BLOIS		3.50 m 11.50 ft	75
CHILLON		3.20 m 10.50 ft	80
SAINT-ANDRE-DE-CUBZAC		3.40 m 11.20 ft	80
B3 SOUTH		2.50 m - 3.40 m 8.20 ft - 11.20 ft	50
SAINT-CLOUD		2.25 m 7.40 ft	130

Fig. 4.II Evolution of segment cross-section and weights.

- (c) Contrary to the box section the ultimate moment capacity of the open section depends more on the concrete strength (see Appendix B).

For bridges up to 40 feet (12 m) width a simple box with two webs and one overhang on each side is satisfactory. The length of each overhang can be equal to half the section of the box.

For wider decks the section will comprise two or more boxes with a common top slab and two corbels. Because the double box (three webs with two overhangs) is more complicated for casting and formwork it is used less.

4.3 Prestressing Steel Layout

In the general case of a continuous bridge there are five groups of prestressing steel which may be required.

- (a) The first group (Fig. 4.12) resists the negative moments of the deck during the construction and after the bridge is completed.

The steel is placed in the top slab in different patterns depending on the prestressing system used.

PREVIOUSLY COPYRIGHTED MATERIAL,

GRAPH ON LEAF 44,

NOT MICROFILMED

COPYRIGHT: GOUVERNMENT DU QUEBEC
MINISTERE DES TRANSPORTS
DIRECTION GENERALE DU GENIE

GRAPH OF GRAND'MERE BRIDGE

So, if the strand (Freyssinet, etc.) or wire (BBR, etc.) system is used, since the effective force of each unit is large, the required number of strands or wires is small and they can be concentrated in the upper region of the webs. They run parallel to the top slab and are anchored in the lower third of the webs by a downward parabolical deviation. In this case the vertical component of the prestressing force helps to reduce the section shear forces.

If the bar system is used (Dywidag system for example), the required number of bars is high due to the lower effective force of each unit, and they are spread all over the top slab width. They run parallel to the webs and they are either anchored at the end of the segments without any deviation (Fig. 4.13), or in the webs by a parabolical deviation (fishbone pattern) (Fig. 4.14).

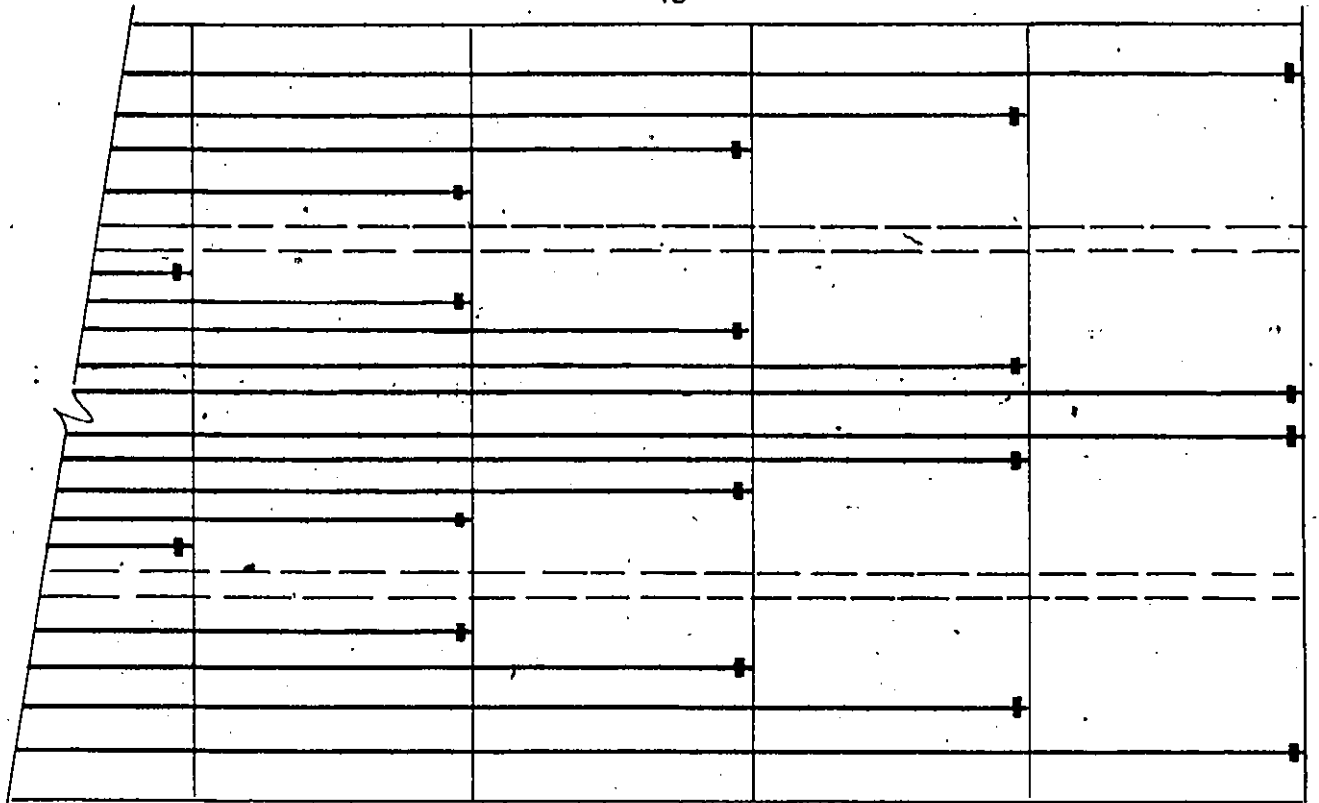


Fig. 4.13 Parallel layout of the prestressing steel

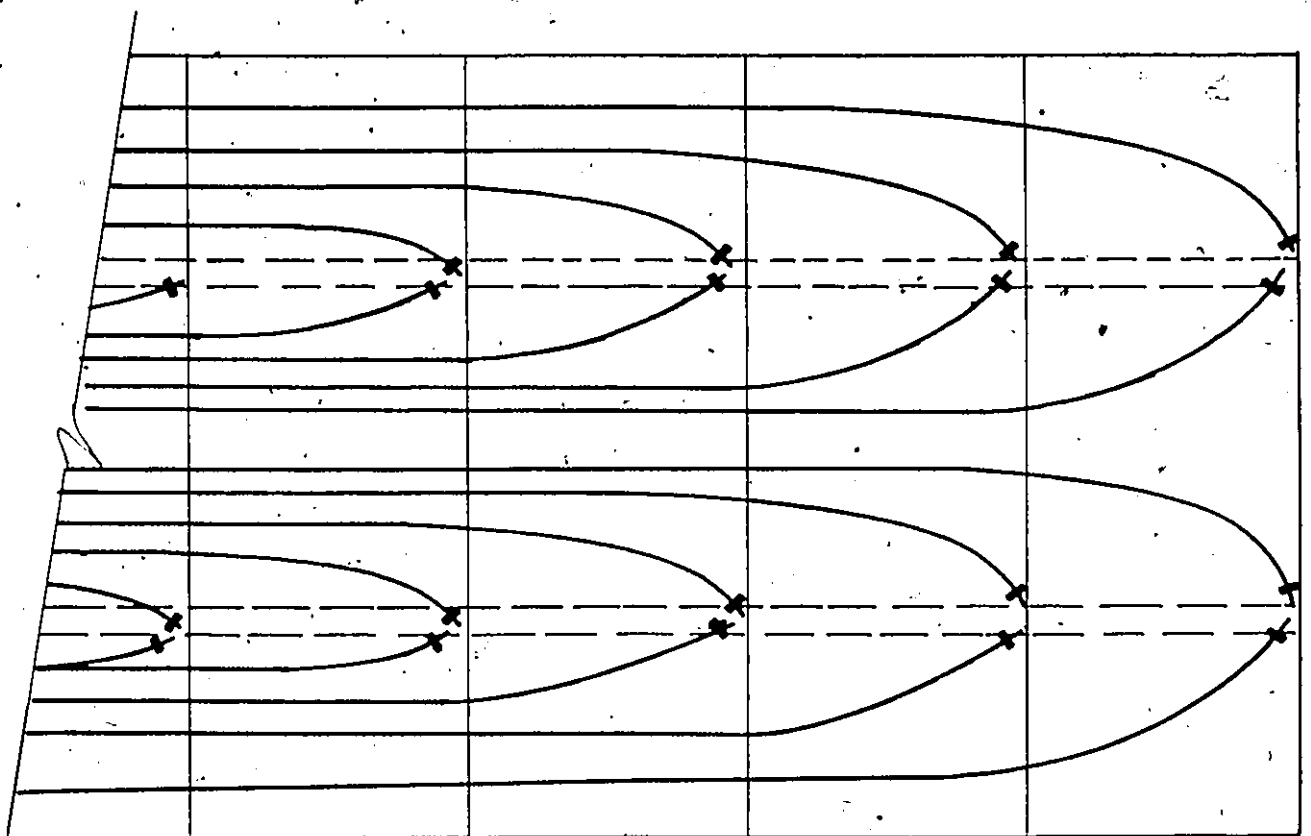


Fig 4.14 "Fishbone" layout.

- (b) The second group (Fig. 4.15) provides the shear strength of the deck during construction as well as after and is comprised of prestressed stirrups placed vertically or inclined at 45° .

This group of prestressing steel is not obligatory because it depends on the prestressing system used. Thus, in the case where the prestressing units are anchored in the bottom part of the webs and with a box section of variable height, the shear force can be greatly reduced and the use of mild steel stirrups may be sufficient.

- (c) The third group (Fig. 4.16) assures the continuity of the two cantilevers and takes the positive moment produced in the middle of the spans. It is located in the bottom slab and the webs.

- (d) The fourth group (Fig. 4.17) provides the capacity in positive moment of the exterior spans near the abutments and the layout is similar to the third group.

PREVIOUSLY COPYRIGHTED MATERIAL,
GRAPH ON LEAF 48,
NOT MICROFILMED

COPYRIGHT: GOUVERNMENT DU QUEBEC
MINISTERE DES TRANSPORTS
DIRECTION GENERALE DU GENIE

GRAPH OF GRAND'MERE BRIDGE

PREVIOUSLY COPYRIGHTED MATERIAL,
GRAPH ON LEAF 49,
NOT MICROFILMED

COPYRIGHT: GOUVERNEMENT DU QUEBEC
MINISTERE DES TRANSPORTS
DIRECTION GENERALE DU GENIE

GRAPH OF GRAND'MERE BRIDGE.

PREVIOUSLY COPYRIGHTED MATERIAL,
GRAPH ON LEAF 50,
NOT MICROFILMED

COPYRIGHT: GOUVERNEMENT DU QUEBEC
MINISTERE DES TRANSPORTS
DIRECTION GENERALE DU GENIE

GRAPH OF GRAN'MERE BRIDGED

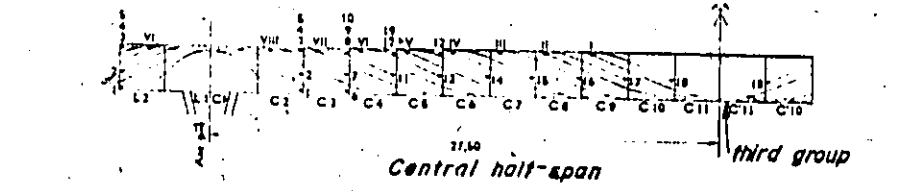
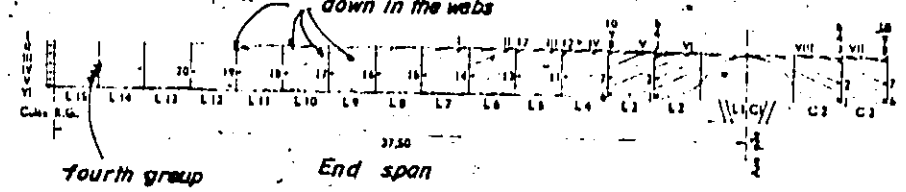
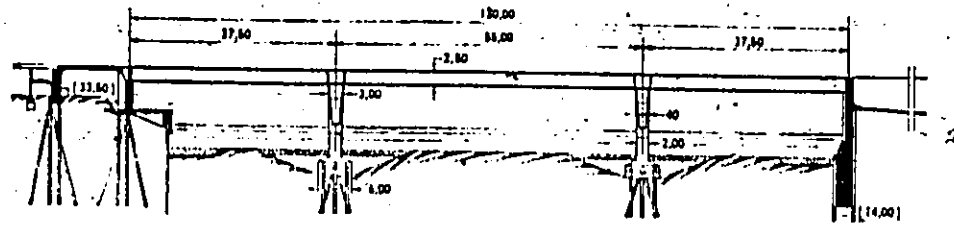


Fig.4.17 Choisy-le-Roi Bridge, France

- (e) The last group (Fig. 4.18) carries the transverse moments created in the top slab and it may not be required if the mild steel reinforcement is adequate.

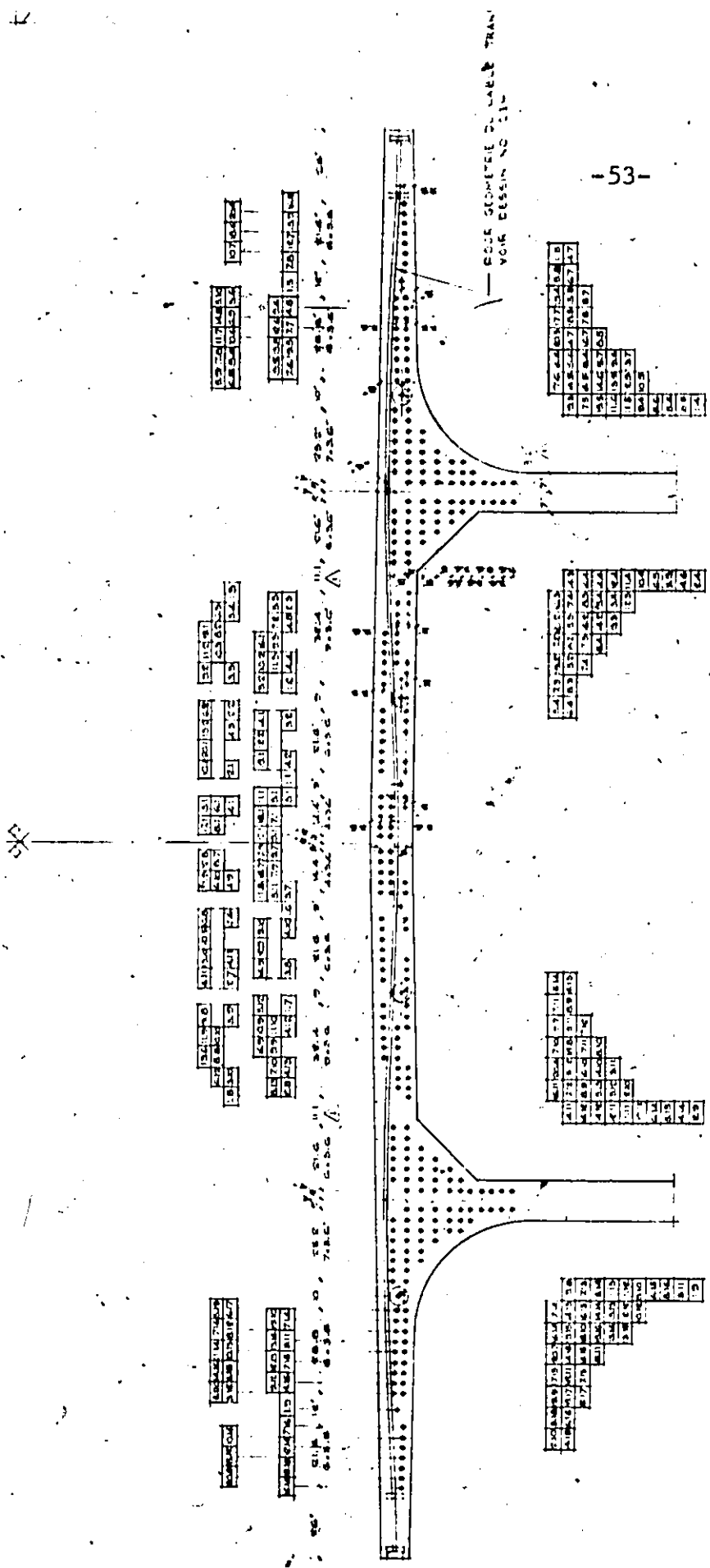


Fig. 4.18 Grand'Mère Bridge. Transverse posttensioning of the top slab.

CHAPTER 5
CONSTRUCTION

5.1 General

The construction of a bridge by the cantilever method can be divided in two phases: the casting of the segments and the placing of them to form the deck.

These two phases can be either combined in a single one or completely separated, depending on the construction system.

The casting is done in steel or wood forms with the mild steel and the ducts for the prestressing steel already in place. In the bar and wire post-tensioning systems, the prestressing elements are already placed in the ducts before pouring the concrete.

The placing of the segments can be done:

- (a) By suspending the casting forms from the already finished part of the deck. The two phases then are combined to one, the casting and placing being done at the same time.

- (b) By erecting prefabricated segments. The two phases are completely separated, the casting of the segments being done at the prefabrication area.
- (c) By casting the whole deck on shore and pushing it gradually towards the river. This method is similar to the first one but the sequence of the construction is completely different.

It is clear that since there are many construction possibilities, a close collaboration must exist between the general contractor and the consulting engineer in order to examine step by step all the details and construction methods and verify if there is any need of modification in the various parts of the structure.

As a result:

- (a) The length of the segments or the prestressing system may be completely different from those chosen in the original design.

The length of the segments can change not only from one construction system to the other but between contractors, depending upon the equipment

available.

In the cast-in-place method, the length of the segments is limited by the form-traveller capacity; in Grand'Mère Bridge for example, the form-traveller was able to accommodate segments of 16 feet in length and weighing up to almost 200 tons.

In the prefabricated method, the segment length is dictated by conditions such as transportation through urban areas, over existing structures, or the capacity of lifting and erection equipment on the site, etc.

The choice of the prestressing system may modify the transverse dimensions of the deck, in order to accommodate the prestressing steel, or the shear design (post-tensioned stirrups, etc.).

- (b) Due to the construction system, an unbalanced condition can result from the two cantilevers during the erection, and probably a new design of the substructure must be done.

- (c) If the erection method chosen by the contractor is different from the one shown on the drawings, a new design will be necessary (see Chapter 7 - Knight Street Bridge).

5.2 Construction by Cast-in-Place Segments

5.2.1 General

The segments are cast in a mould supported by a steel form-traveller moving on rails on the finished part of the deck as the construction progresses.

The form-traveller (Fig. 5.1, Ref. 5.1) is comprised of a system of main lengthwise and secondary crosswise beams from which the exterior and interior formwork, as well as the bottom of the mould and the working platform, are hung.

Each main longitudinal beam (truss beam) has two bearing points on the top slab of the already finished deck, the front point being near the edge of the slab and the second one approximately 30 feet back. As the form-traveller is subjected to overturning moments, the back bearing points must be held down either by concrete blocks

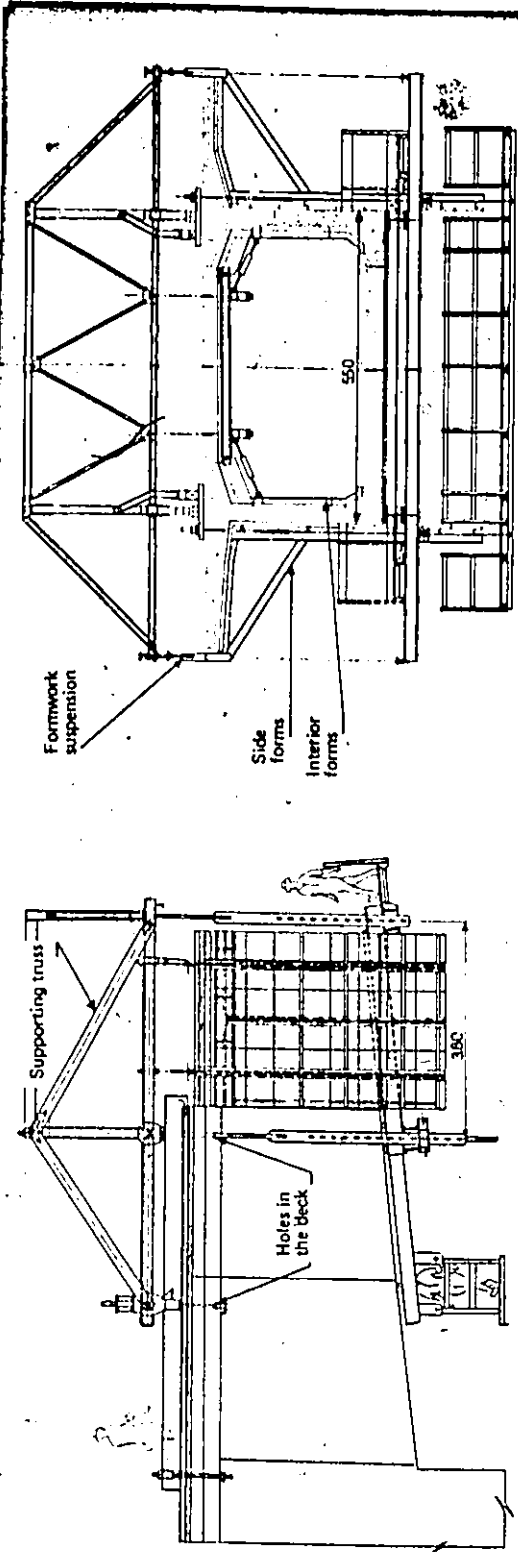


Fig. 5.1 Mobile falsework for cantilever construction.

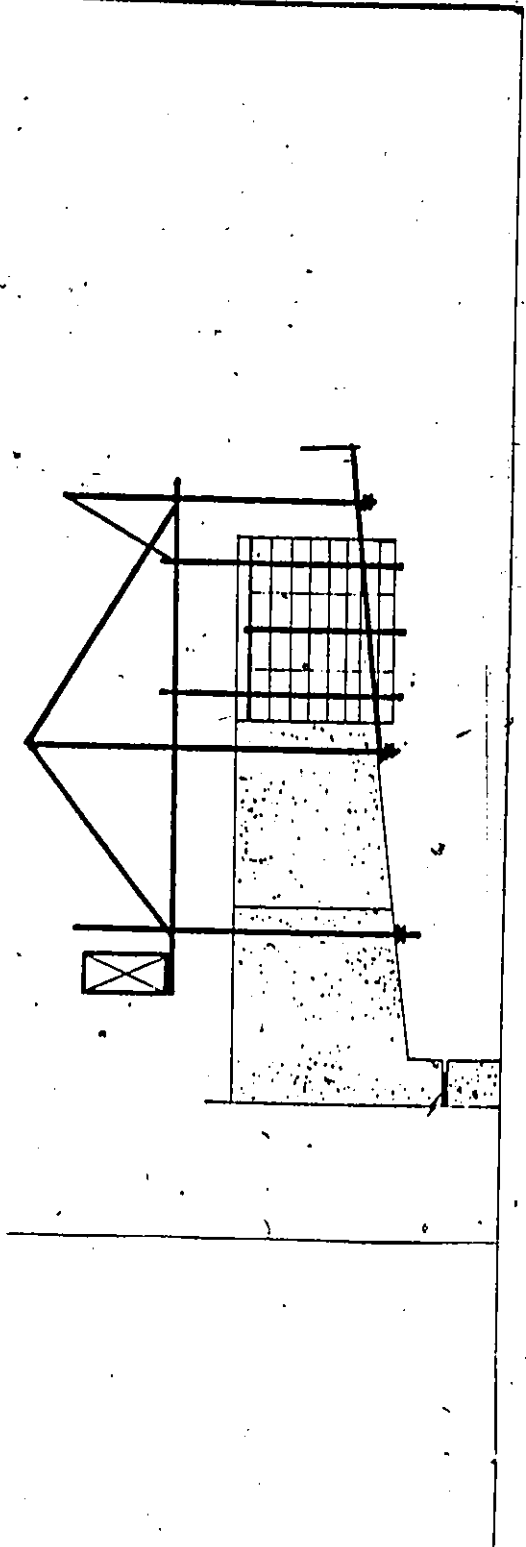


Fig. 5.2 Schematic view of a form-traveller with concrete block and hold-down anchors.

acting as counter-weights (Fig. 5.2, Ref. 5.7) or by anchoring it to the deck by tie-rods (Fig. 5.1). The form-traveller can be completely enclosed for cold weather concreting or steam-cured concrete (Fig. 5.3).

5.2.2 Construction Methods

5.2.2.1 Symmetrical Cantilevering

The main idea, as mentioned previously, is to have a form-traveller on each side of the pier and progress simultaneously, segment by segment (Fig. 5.3). Starting at the top of each pier (Fig. 5.4), the portion above it V_0 and a small element of the deck V_1 and V_1' (starting element) are cast on a scaffolding which, at the same time, will stabilize the hammerhead during the construction. The pier table with the first elements will serve as a platform to assemble the form travellers and provide the first anchor points for them.

If the length of this platform is too small to accommodate the two form-travellers simultaneously, it is necessary to erect one of them on the top of the pier element and cast the first segment of one cantilever. Once

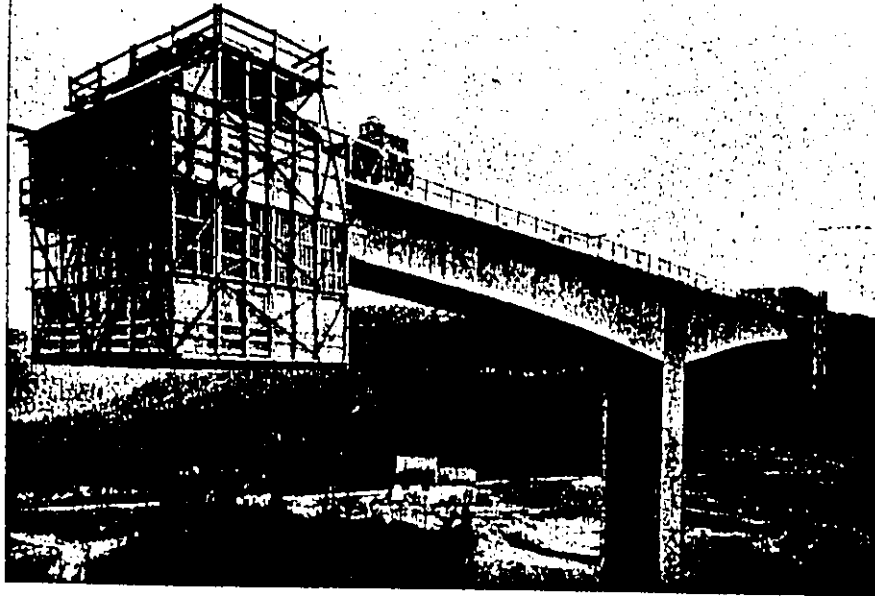


Fig. 5.3 Pyle Bridge, France.

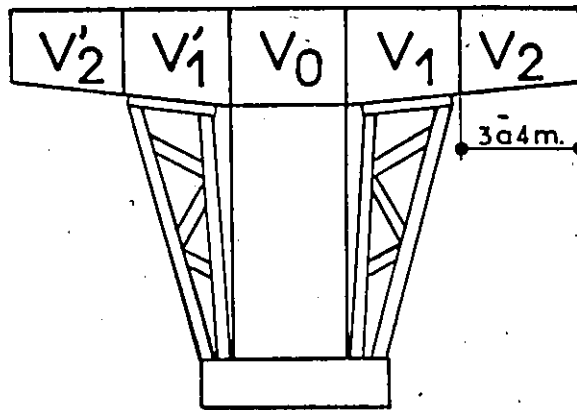


Fig. 5.4 Casting the segments near the pier.

this is finished, the form-traveller is moved forward to cast the next segment, at the same time clearing the deck over the pier in order to assemble the second form-traveller and start the construction of the opposite cantilever (Fig. 5.5, Ref. 5.3).

In such cases, the double cantilever system is unbalanced by at least one segment which can affect the design of the pier and its foundation (see deck stability during construction).

This points out once more that the engineer's final design cannot be completed before the erection method is known.

5.2.2.2 Scaffolding and Cantilevering

It sometimes happens that the symmetrical construction, as described previously, is not necessarily the best solution. Thus, for bridges with decks close to the ground (Fig. 5.6) or with difficult site conditions (Fig. 5.7, Ref. 5.7), it is probably better to cast the exterior spans partially (Fig. 5.8) or completely (Fig. 5.6) on scaffolding and proceed for the other spans with only one cantilever from each bank.

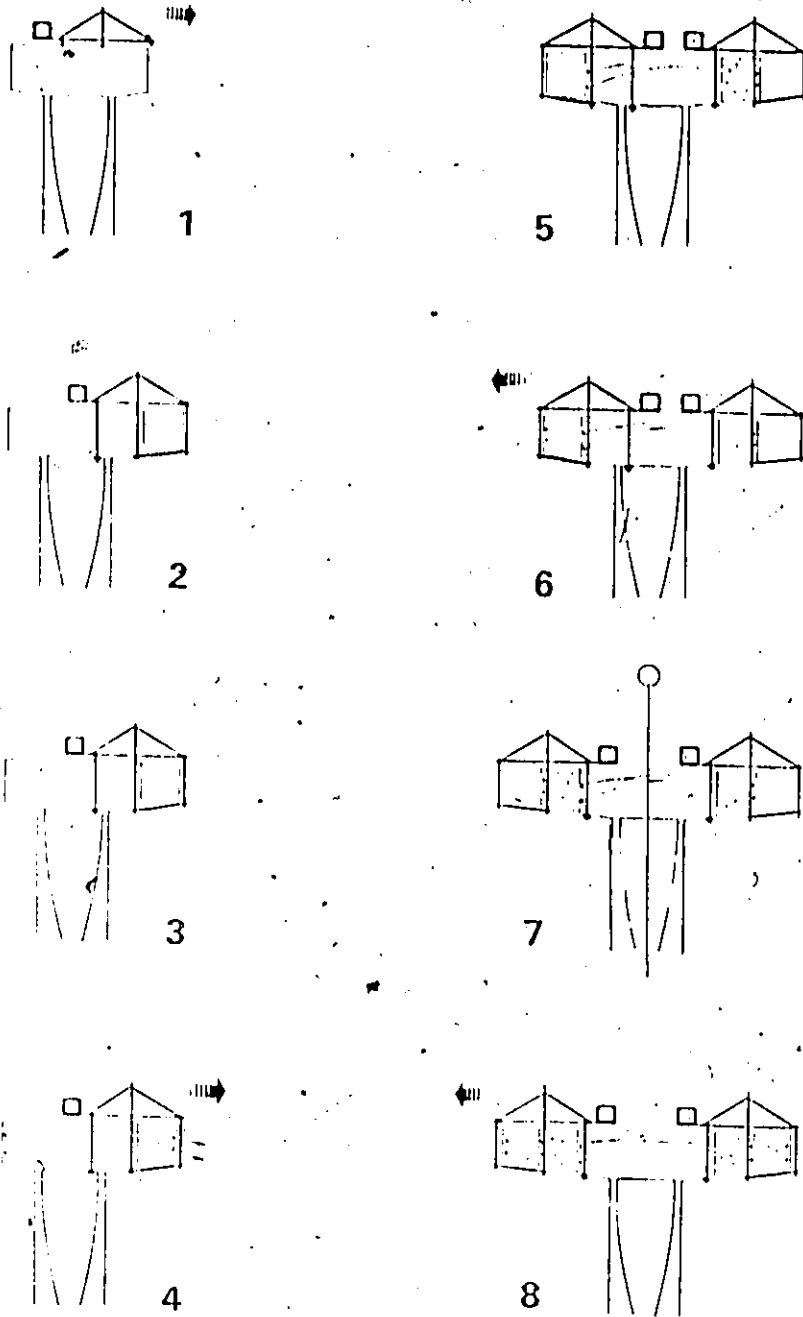


Fig. 5.5 Bosques de Reforma Viaduct, Mexico.
Construction sequences of the first
segments.

PREVIOUSLY COPYRIGHTED MATERIAL,
GRAPH ON LEAF 63,
NOT MICROFILMED

COPYRIGHT: GOUVERNMENT DU QUEBEC
MINISTERE DES TRANSPORTS
DIRECTION GENERALE DU GENIE

GRAPH OF GRAND'MERE BRIDGE

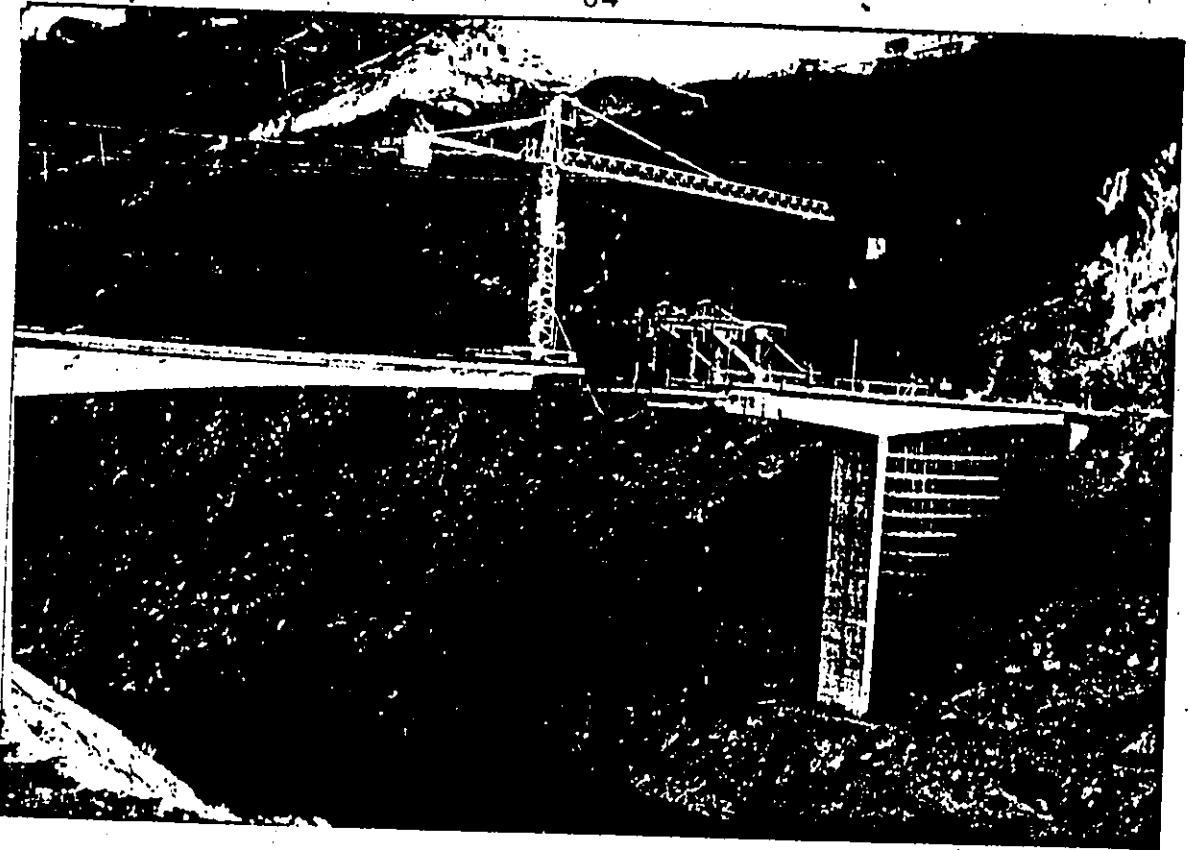


Fig. 5.7 Iznajar Bridge, Spain.
The exterior span was cast on scaffolding.

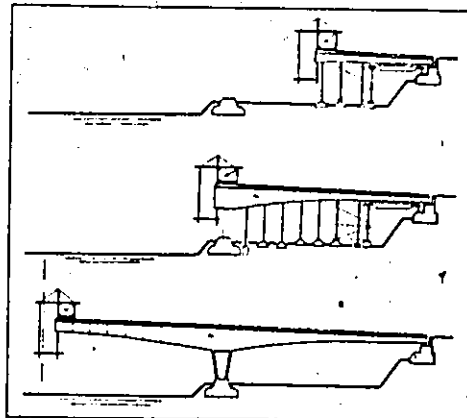


Fig. 5.8 Ringbrücke over the Danube
at Ulm, West Germany.

In this case, the materials are moved over the finished part of the deck, avoiding the use of an expensive cableway.

5.2.2.3. Cantilevering and Cable-Stay

This method, using the cable-stay scheme, consists of erecting the deck with one or two form-travellers. Once the span of the cantilevers becomes too long, the deck is suspended with stays from concrete or steel pylons erected on the top of the piers (Fig. 5.9, Ref. 5.4).

An example where all the above methods were used is the highway and railway bridge built in Caceres, Spain, by the German firm Dyckerhoff & Widmann (Fig. 5.10, Ref. 5.2). The exterior spans over the banks were built on scaffolding and by cantilevering, the second and fourth spans by cantilevering with temporary cable-stay, and finally the middle one by cantilevering. An interesting example of the clever use of the available methods.

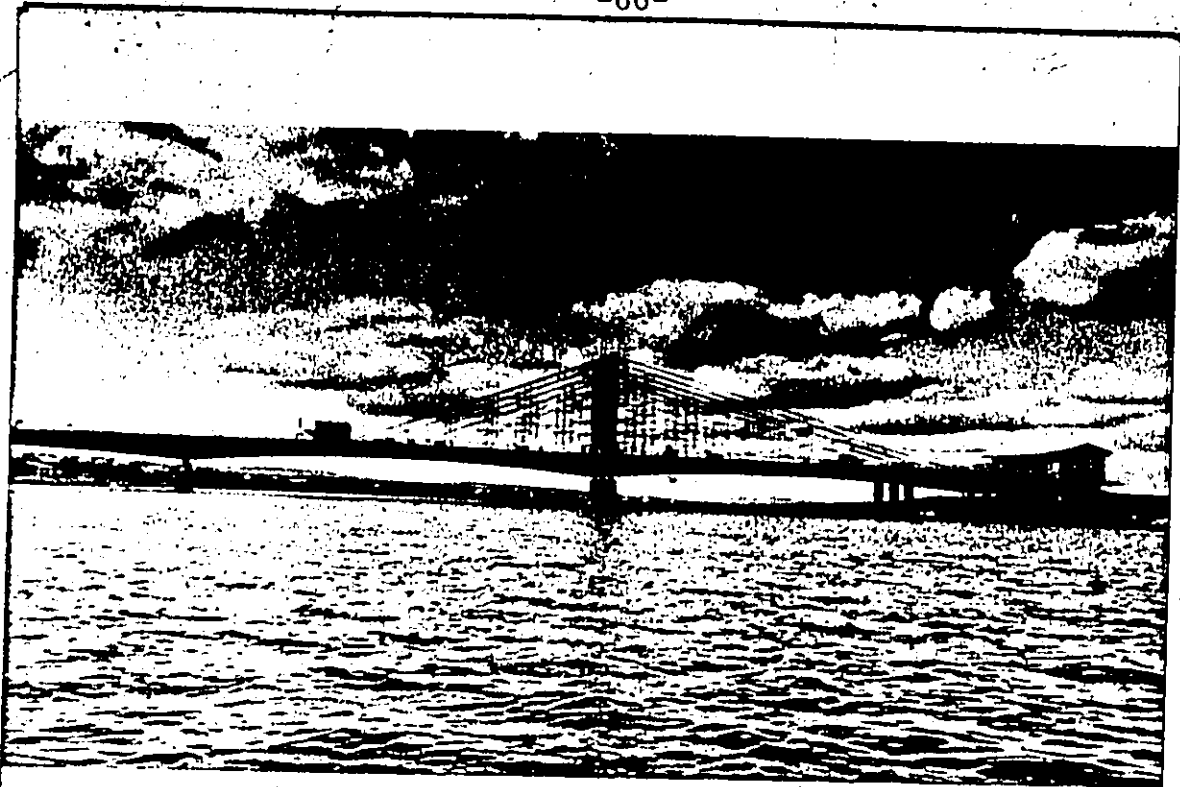


Fig. 5.9 Ounasjoki Bridge on the Arctic Circle, Finland.

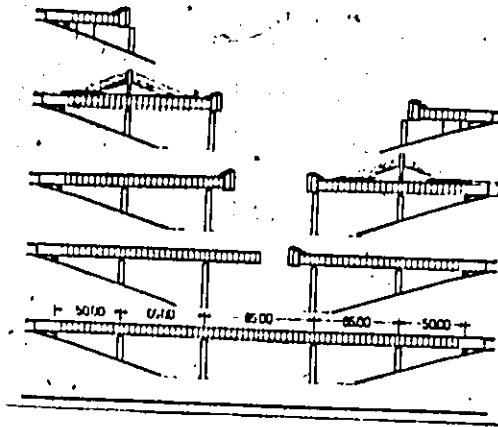


Fig. 5.10 Caceres Bridge, Spain.

5.2.2.4 Cantilevering with an Auxiliary Steel Beam

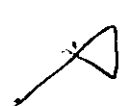
This method consists in casting the deck in the cantilever method but instead of using a form-traveller, the concrete weight is supported by a steel beam anchored on the finished deck.

The beam can be either very long (almost one and a half times the span length) supported by the piers (Fig. 5.11, Ref. 5.5), or small and light supported by the cantilever edge with an overhang of only one segment length (Fig. 5.12, Ref. 5.5).

In the first case, the segment length can be 30 feet, and in the second it can vary between 25 feet and 40 feet.

5.2.3 Construction Phases of a Segment

In general, the construction phases of a segment (Ref. 5.6) are the following:

- (i) Moving of the form-traveller, fixing it in its new position, and adjustment of the interior and exterior formwork.
- 

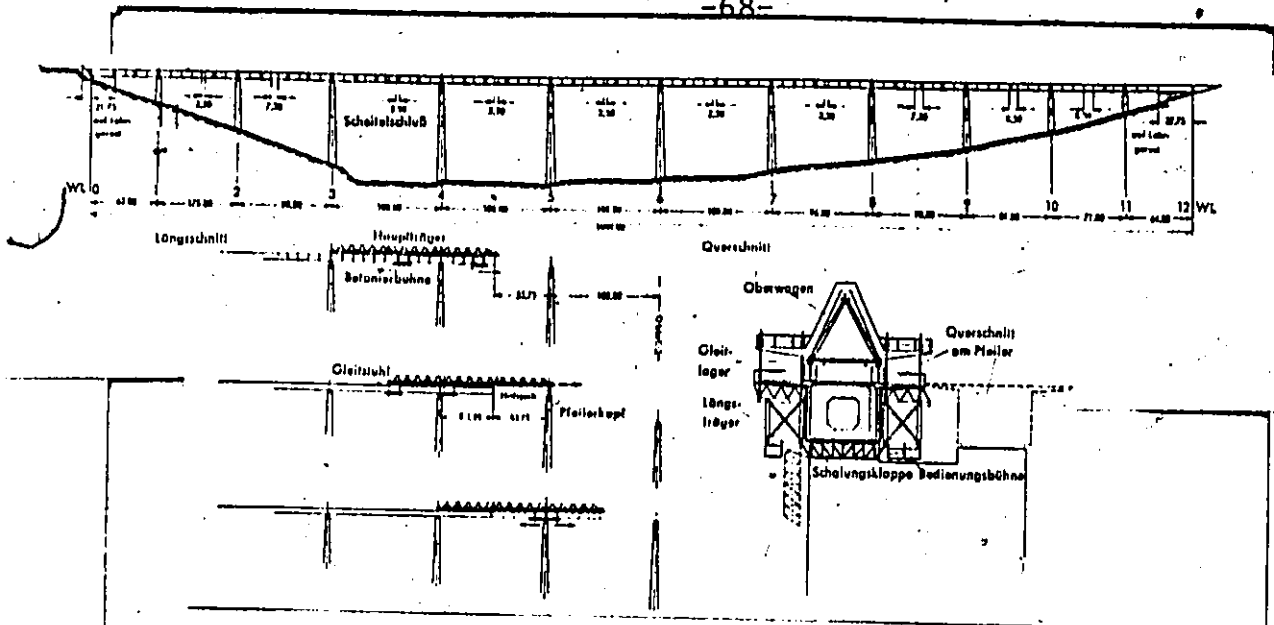


Fig. 5.II Auxiliary steel girder of the Siegtal Bridge.

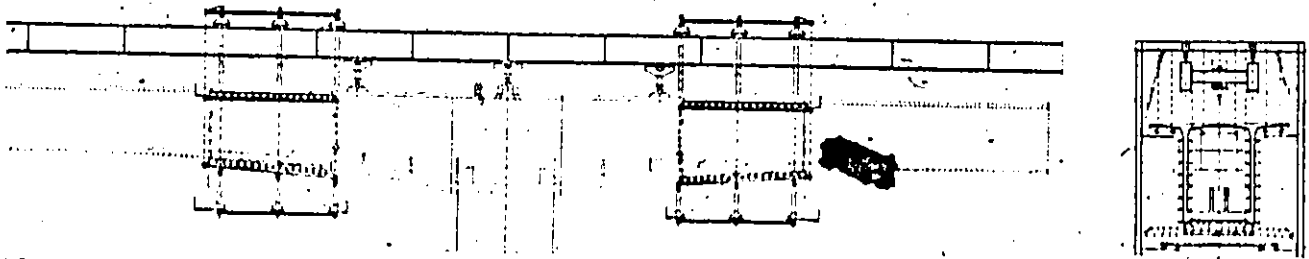


Fig. 5.I2 Light auxiliary bridge for cantilever sections of 25 to 40 feet.

- (ii) Placing of the mild steel and the prestressing ducts, if they exist, on the bottom slab and the mild steel in the webs.
- (iii) Concreting of the bottom slab between the webs.
- (iv) Placing the longitudinal prestressing steel anchored in the segment under construction and the remaining mild steel in the webs.
- (v) Placing all the ducts located in the webs and passing through this segment.
- (vi) Placing the interior formwork.
- (vii) Placing all the ducts of the longitudinal and transverse prestressing steel located in the top slab as well as the mild steel.
- (viii) Concreting of webs and of the top slab (at the same time if the quantities are small).

The placing of the concrete in the webs must be started from the end of the mould and proceed towards the finished part of the deck in order to

avoid the cracks in the fresh concrete due to the deflection of the form-traveller (Fig. 5.13).

- (ix) Slacking of the interior formwork to allow the free shrinkage of the concrete.
- (x) Injection of the longitudinal ducts anchored in one or more segments. This operation can be performed at any convenient time.
- (xi) Tensioning of the transverse prestressing steel (Fig. 5.14). This operation can be delayed if the mild steel of the top slab is adequate for the bending moments due to dead and construction loads.
- (xii) Slacking of the exterior formwork.
- (xiii) Tensioning of the prestressed stirrups and longitudinal prestressing steel (Fig. 5.14) anchored in this segment.
- (xiv) Preparation for the moving of the form-traveller to the next segment.

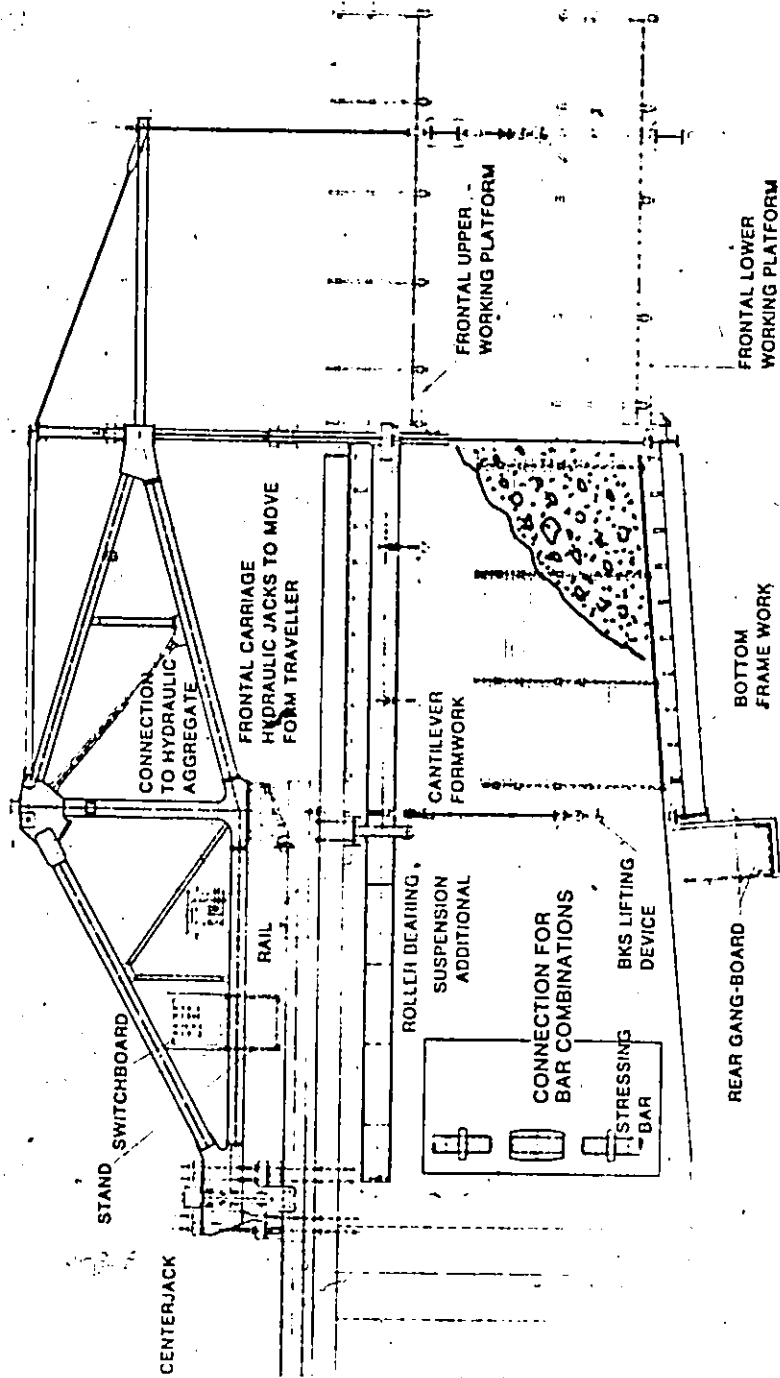


Fig.5.I3 Placing of the fresh concrete in the webs.

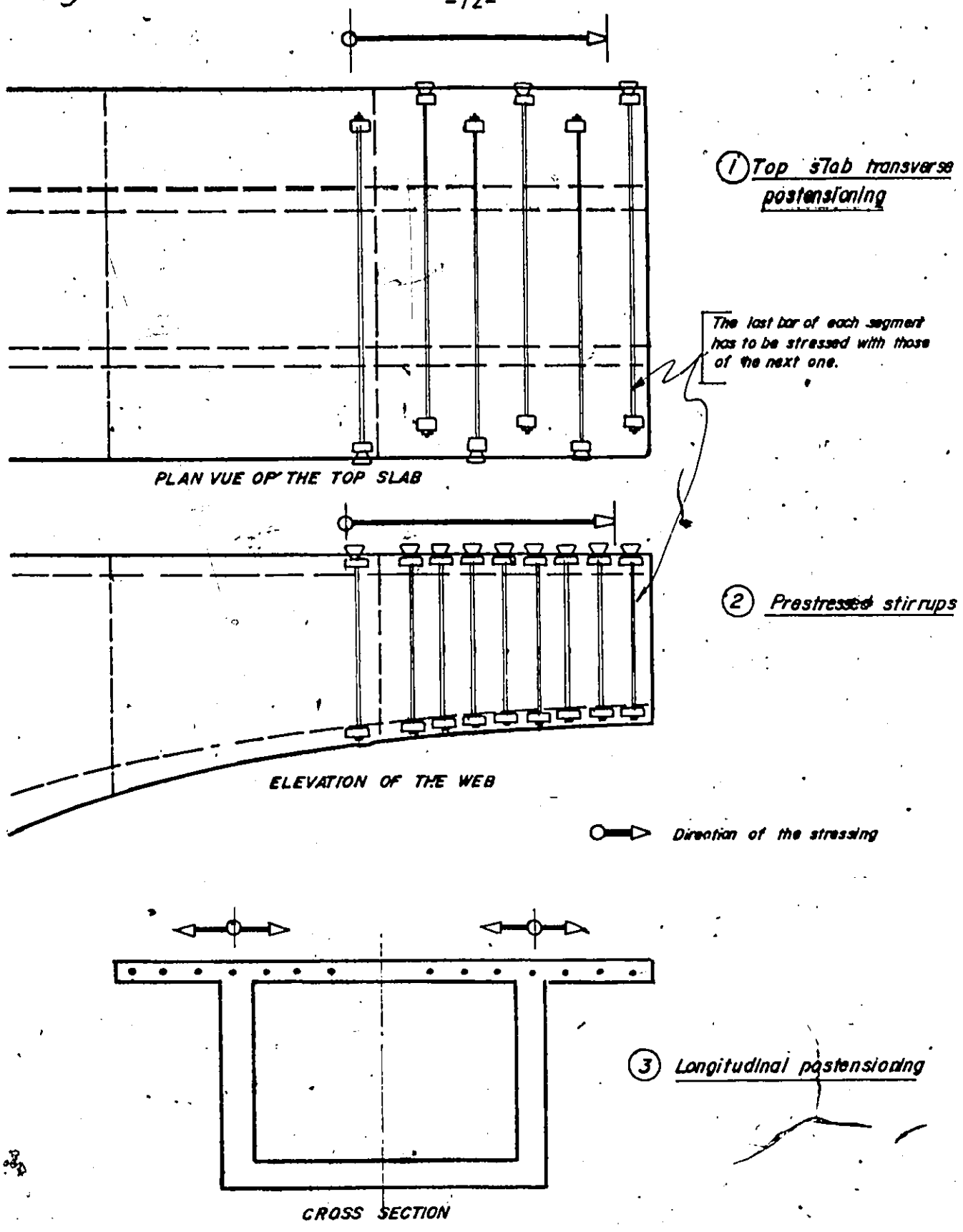


Fig. 514 Grand'Mere Bridge. Stressing sequences of a segment.

The normal duration of construction permits a complete cycle in five working days per week.

- (a) One day for stressing the previously concreted segment, removing the form-traveller and placing it in its new position (Monday).
- (b) Three days for the placing of the mild and prestressing steel (Tuesday to Thursday).
- (c) One day for pouring (Friday).
- (d) Two days for the concrete hardening (Saturday and Sunday).

This cycle corresponds to a progress of $\frac{15 \text{ feet} \times 2}{5} = 6$ feet per working day for two form-travellers.

Greater progress can be achieved with more form-travellers (construction from more than one pier at a time), or by reducing the described cycle by accelerating the concrete hardening (steam curing, etc.), or by using prefabricated bulkheads in the anchor zones at the end of the segments which permits an earlier post-tensioning of the cables.

In Grand'Mère Bridge, for example, the whole segment was cast at the same time and a vapor curing was used in order to obtain a high compression strength in the concrete very quickly. A few hours later (when the concrete reached a 2200 psi compressive strength) a partial transverse and longitudinal prestressing was applied and the form-traveller was moved to its new position. Once it was completely installed, the total post-tensioning of the segment was performed since during that time the concrete strength had reached the minimum required.

5.3 Construction by Erecting Prefabricated Segments

5.3.1 General

This method consists of erecting the deck with prefabricated segments of 10 feet to 15 feet long, the length being limited by the capacity of the lifting equipment.

The elements are cast in a yard or in the factory and transported on the site for erection or stored until ready for use.

The advantage of this method is the speed of production which should be matched with an erection speed of the same order.

But, according to the classical prefabrication methods, a gap exists between the different elements which must be filled or caulked with mortar during the erection. If the same method was used for the bridges erected by cantilevering, the whole advantage of the prefabrication would be lost due to the waiting time between the erection and the stressing of an element to allow for the setting of the mortar.

To overcome this drawback "thickless" joints called conjugate or match-cast joints are used which ensure a proper fit during erection.

These joints are obtained by casting one segment after the other, using the vertical surface of the previously finished segment as the form for the opposite face of the next segment (match-cast segments).

5.3.2 Prefabrication Methods

The prefabrication methods can be divided in two main categories.

(a) Precasting on a "prefabrication bench" all the elements of the bridge. These include the sections over the pier and the segments of the two cantilevers (long line method).

(b) Casting in one or more moulds all the segments of the deck (short line method).

In the first method, the formwork is moved as the prefabrication progresses, although in the second, one or more moulds are stationary and only the cast segments are moved.

5.3.2.1 Precasting on a Prefabrication Bench (Long Line Method)

This method consists of precasting on a falsework a complete hammerhead comprising the pier-table and the first segments of the cantilevers on each side of it.

If, for economical reasons or space limitations, the falsework has to be reduced to only a half hammerhead (Fig. 5.15, Ref. 5.8), that is the pier-table and one cantilever, before starting to cast the second half, then the pier-table has to be turned around by 180° in order to obtain the second match-cast joint.

On the falsework erected in the prefabrication yard, the formwork of the intrados of the bottom slab of the hammerhead is fabricated. The formwork for the webs and top slab is fabricated for one segment only and moves as the production progresses. These moving parts of the formwork have to be carefully designed in order to permit an easy positioning, removing and advancing along the whole line of production.

The prefabrication phases are the following:

- (a) Using a special interior formwork (due to the particular inside shape) (Fig. 5.16, Ref. 5.9) and a normal exterior formwork the pier element is cast.

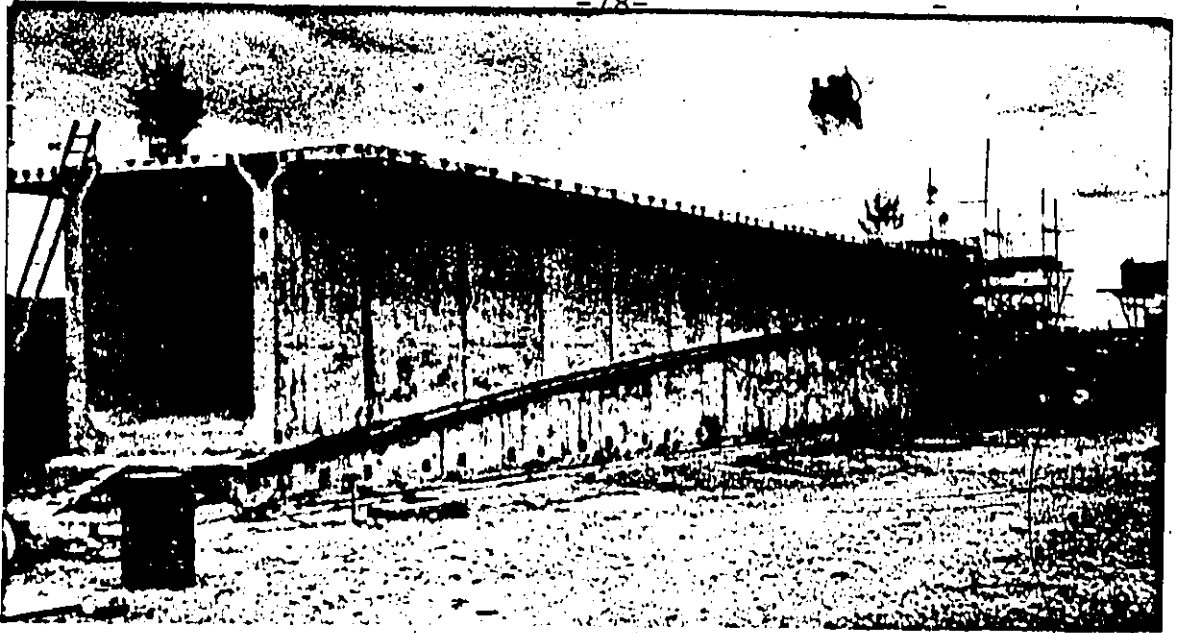


Fig. 5.15. Precasting of elements on a template form.

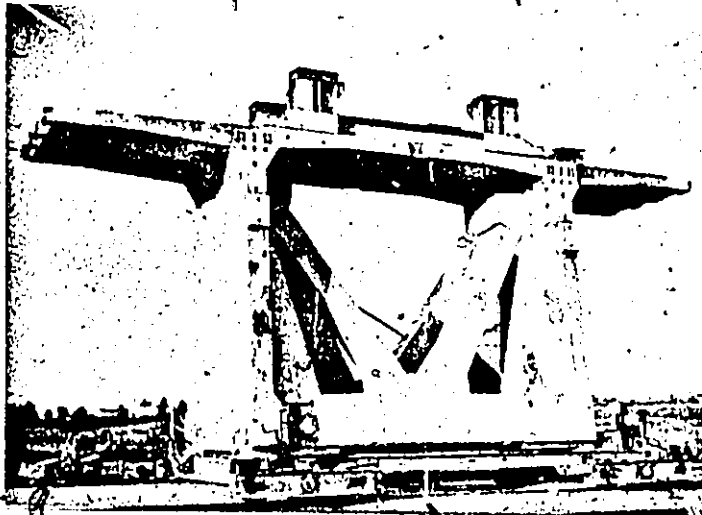
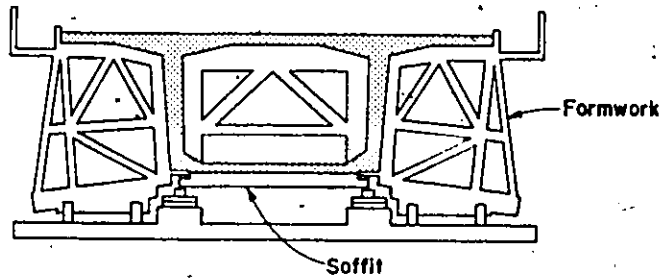


Fig. 5.16 Oléron Viaduct, France. Pier element.

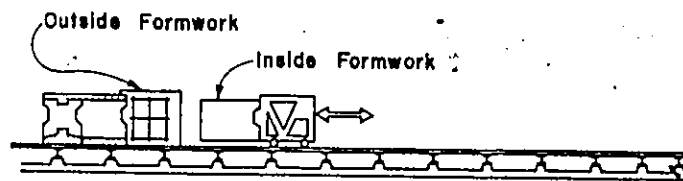
(b) The adjoining segments are cast against the one previously finished, using them as a bulkhead for their common joint. It should be noted that before concreting this bulkhead has to be coated with a bond-breaking agent in order to facilitate the separation of the two elements.

(c) Transportation to the storage area of the completed segments and not required as bulkheads. During storage, special attention has to be given in order to minimize any deflection, twisting or damage of the units. Stockpiling should be limited to avoid excessive direct or eccentric forces or ground settlements.

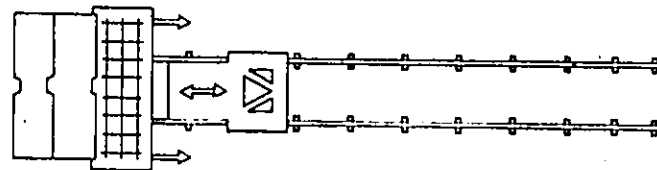
A typical section is shown in Fig. 5.17 (Ref. 5.10). As can be seen, the inside as well as the outside forms move on rails. The exterior formwork has jacks for removing it, while the interior one is telescopic. The removal and transportation to the storage area is done either by special movable cranes striding over the bench (Fig. 5.18, Ref. 5.9), or ordinary ones (Fig. 5.19).



Cross section of formwork using long-line method.

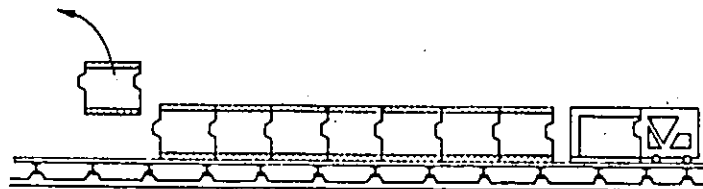


ELEVATION

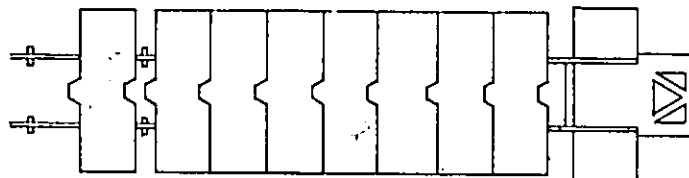


PLAN

Start of casting (long-line method).



ELEVATION



PLAN

After casting several segments (long-line method).

Fig. 5.17 Construction sequences of the long-line method.

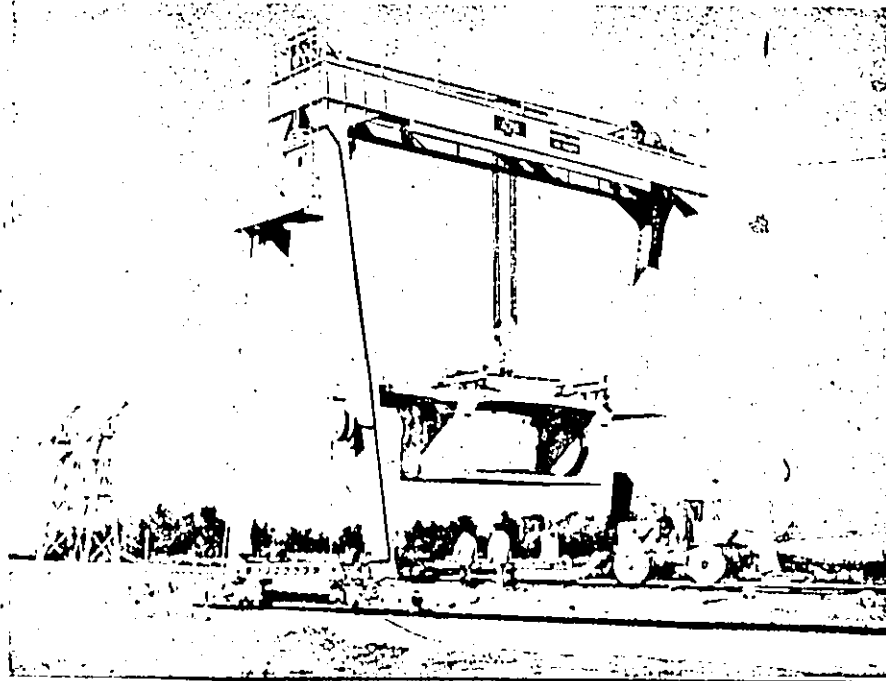
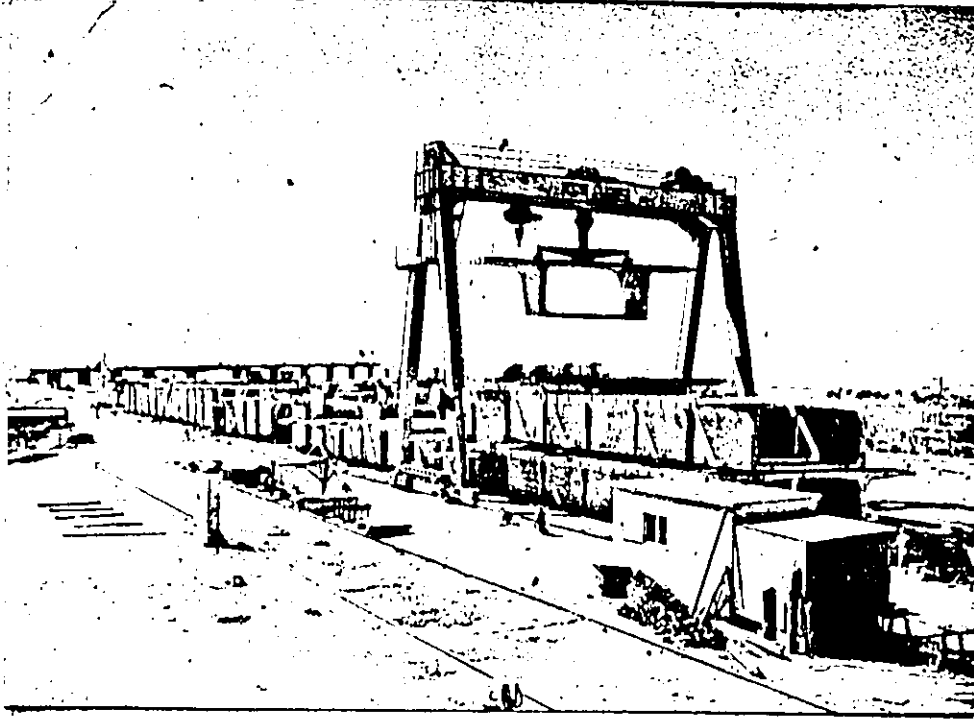


Fig. 5.18 Oleron Viaduct, France.
Transportation of the segments.

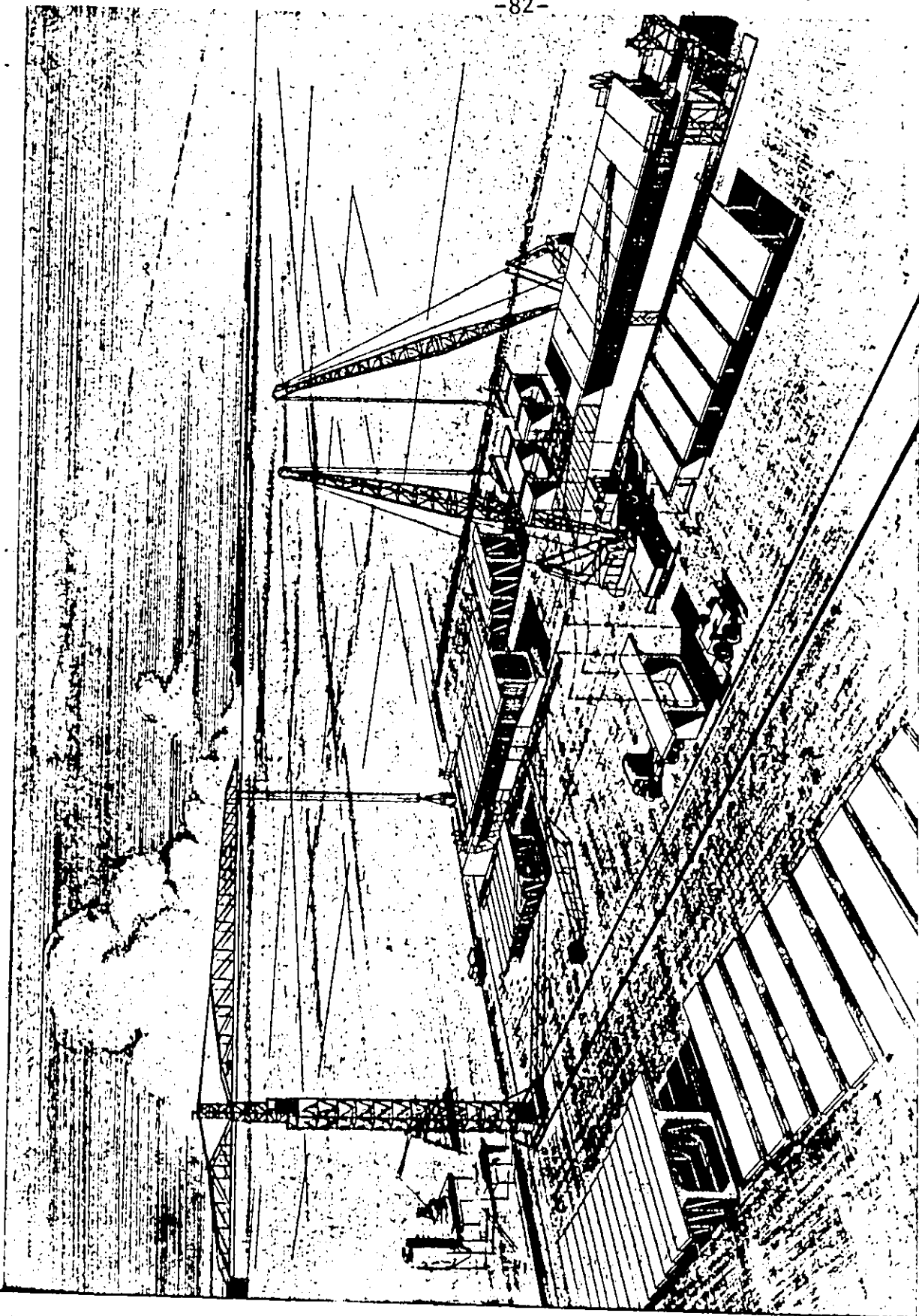


Fig. 5. 19 Schematic view of a prefabrication yard.

The concrete is usually steam cured, although electrical wires can be used, and it is easy to prefabricate two segments a day.

The production can be accelerated to four segments a day by using two sets of formworks and two work crews, especially if the deck has a constant depth.

It is an expensive method because it requires fixed installations whose size increases with the hammerhead length. But it guarantees a perfect line for the intrados which is very important when the horizontal and vertical profiles of the deck are complicated (curved bridges of variable height).

Another drawback, if the hammerheads are not identical, is the necessity to adjust the scaffolding and the formwork of the intrados each time before starting another cantilever.

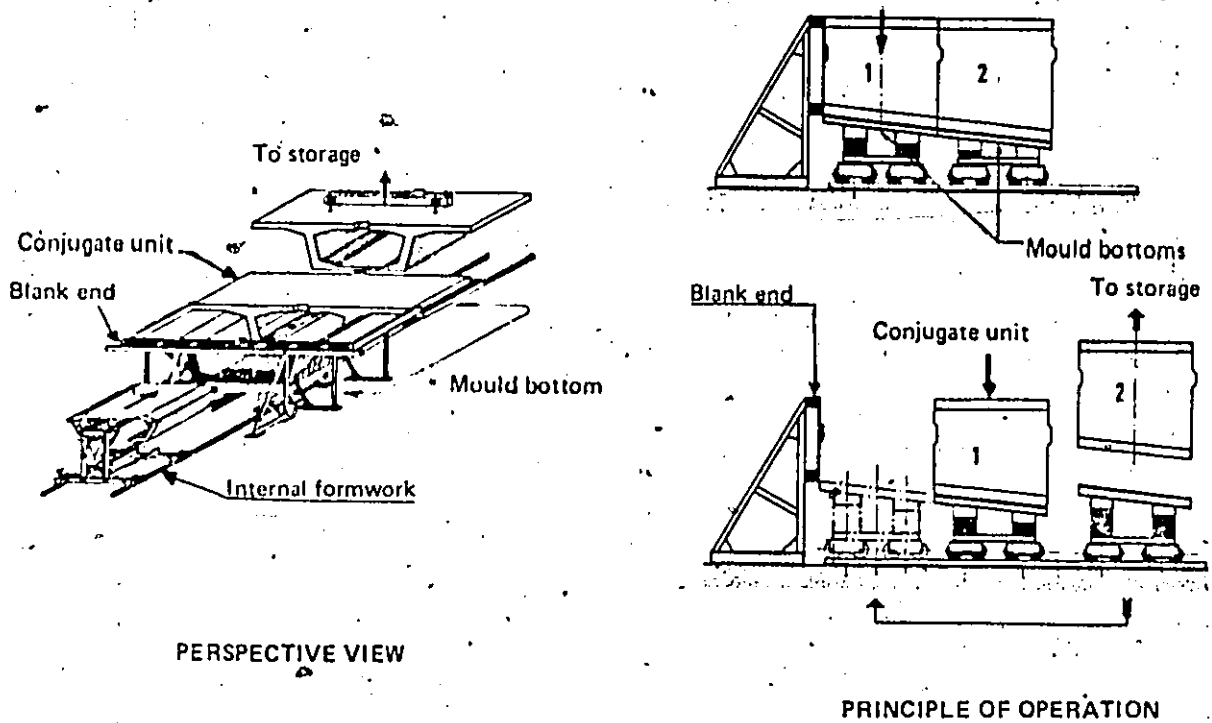
5.3.2.2 Precasting in Stationary Moulds (Short Line Method)

The segments are cast in one or more stationary moulds called prefabrication cells. Each hardened segment is moved to a new position so that it acts as a bulkhead

for the next one (Fig. 5.20, Ref. 5.1). The formwork is designed in a way that permits any possible adjustment whether vertical or horizontal (Fig. 5.21, Ref. 5.10). Each cell is composed by one fixed part and two movable parts. The fixed part is the outside lateral formwork and the bulkhead at the end of the segment (blank end). The movable parts are the interior formwork and the vertical face of the previous precast element which, placed against the fixed formwork, serves as a-form for the new segment, thus completing the match-cast joint.

Once the fixed part is in place, the construction phases are the following:

- (a) Placing of the mild steel and all the ducts for the prestressing steel.
- (b) Placing of the interior formwork.
- (c) Adjustment of the previous cast segment.
- (d) Concreting after which the formwork is removed once the concrete is set (around 10 hours).



PERSPECTIVE VIEW

PRINCIPLE OF OPERATION

Fig. 5.20 Scheme of a precasting cell.

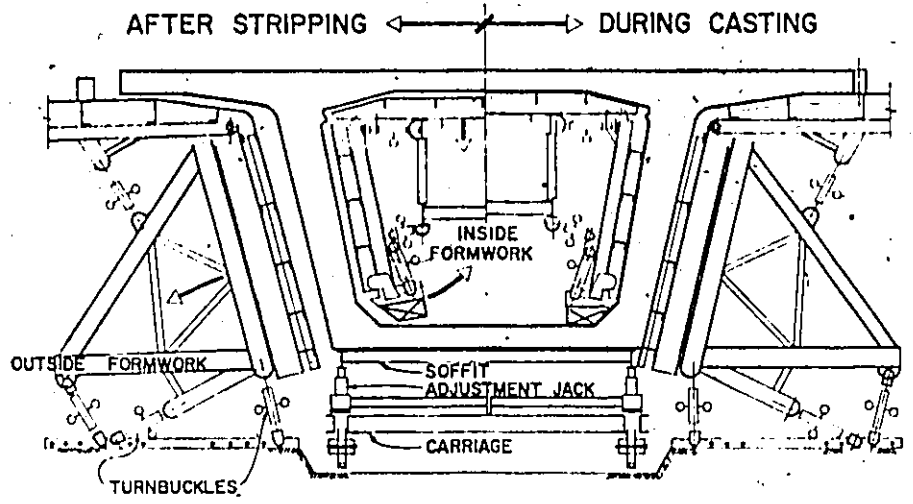


Fig. 5.21 Formwork for the short-line method

- (e) Transportation and storage of the previous segment.
- (f) Moving of the new one in order to be used as a form for the next segment.

A different method was used for the prefabrication of the Parc de Prince Stadium in Paris and the Olympic Stadium in Montreal.

This method consists in casting the segments in prefabrication cells with vertical axes (instead of a horizontal one). This method facilitates the placing of the concrete (all the ducts are vertical), but the rotation of the segment to the horizontal position requires a special device more complicated than a simple crane (Fig. 5.22, Ref. 5.1).

Another method used at Chelcpichinsky Bridge in Russia, consists in using "assembling cells". The different elements of a segment (top and bottom slab, webs) are prefabricated and brought to these cells where they are assembled by concreting the joints (Ref. 5.21).

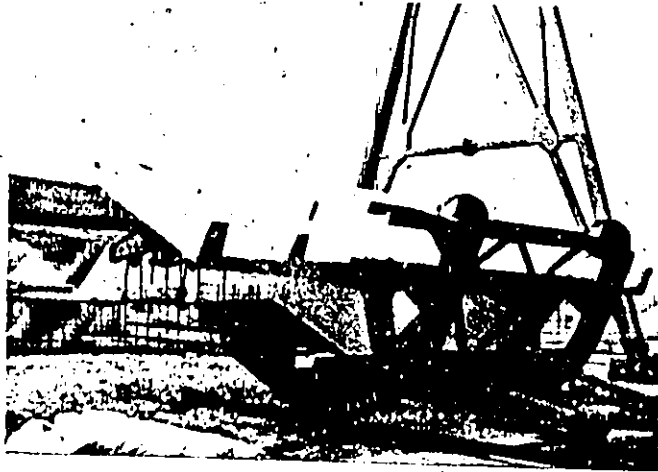


Fig. 5.22 Turning a segment.

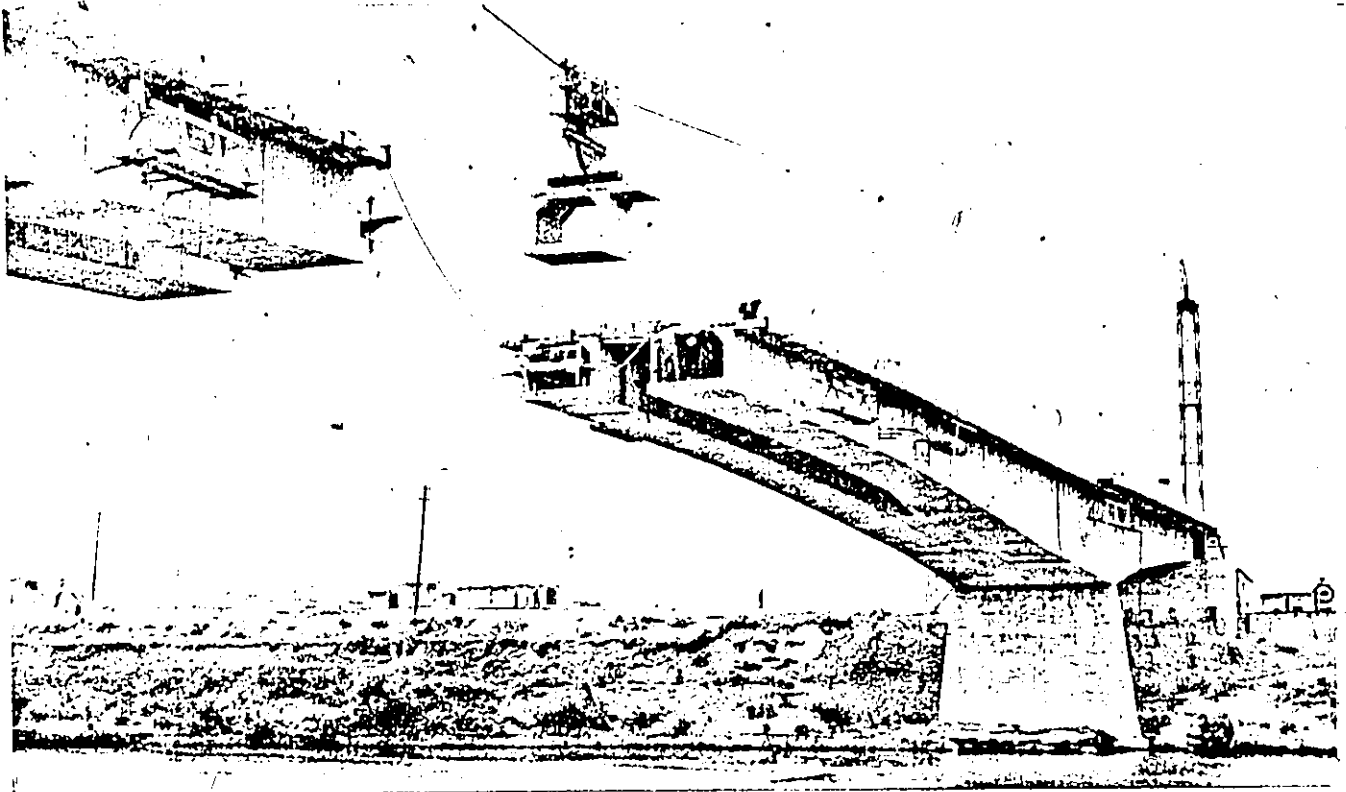


Fig. 5.23 Castejon Bridge over the Rio Erbo, Spain, 1968.

5.3.3 Transportation and Erecting of the Segments

The transportation and erection methods vary according to the characteristics of the site. Some of these methods are:

- (a) Transportation of the segments on the ground or on the water, and erection by a ground or floating crane located outside the deck.
- (b) Transportation as in (a) but erection by a lifting device located on the deck:
- (c) Transportation and erection using special gangways.
- (d) Transportation on the ground, on the water or on the deck itself, and placing the segment with a special launching device.
- (e) Transportation and placing with a cableway (Fig. 5.23, Ref. 5.1).

The first method consists in transporting the segments on trucks or barges and using a fixed or movable crane for the erection (Fig. 5.24, Ref. 5.1).

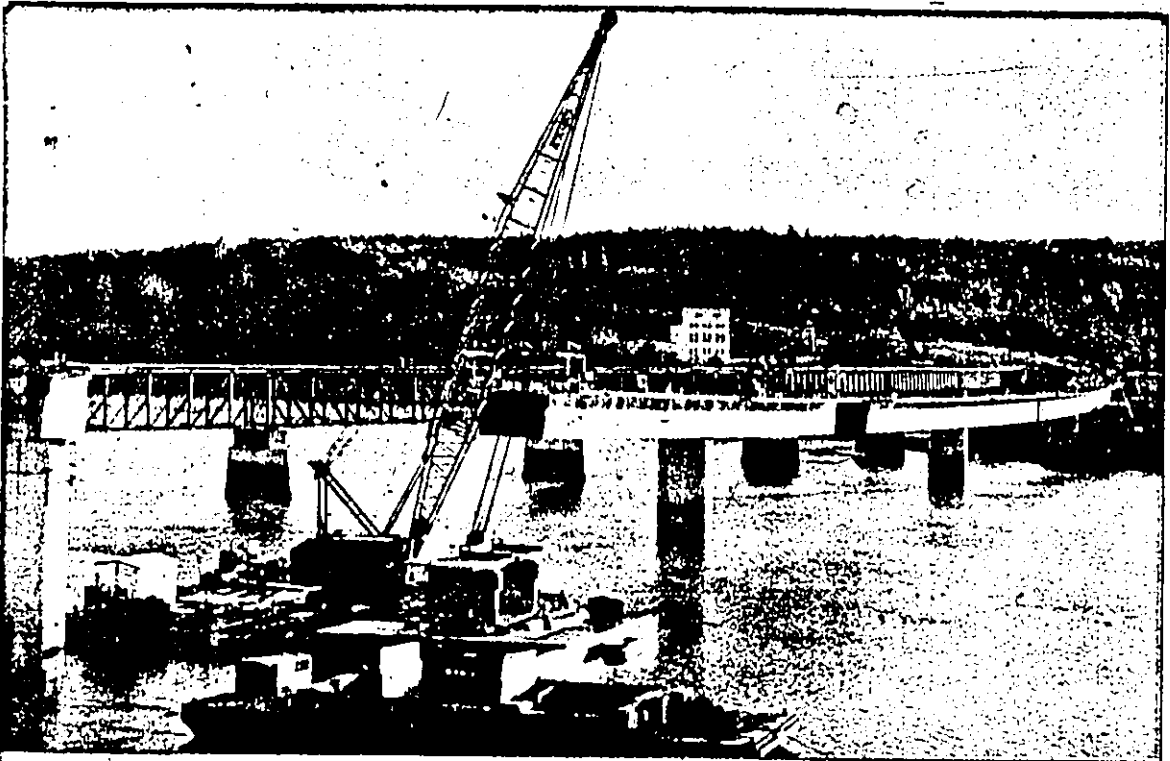


Fig. 5.24 Bear River Bridge, Nova Scotia, Canada, 1972.

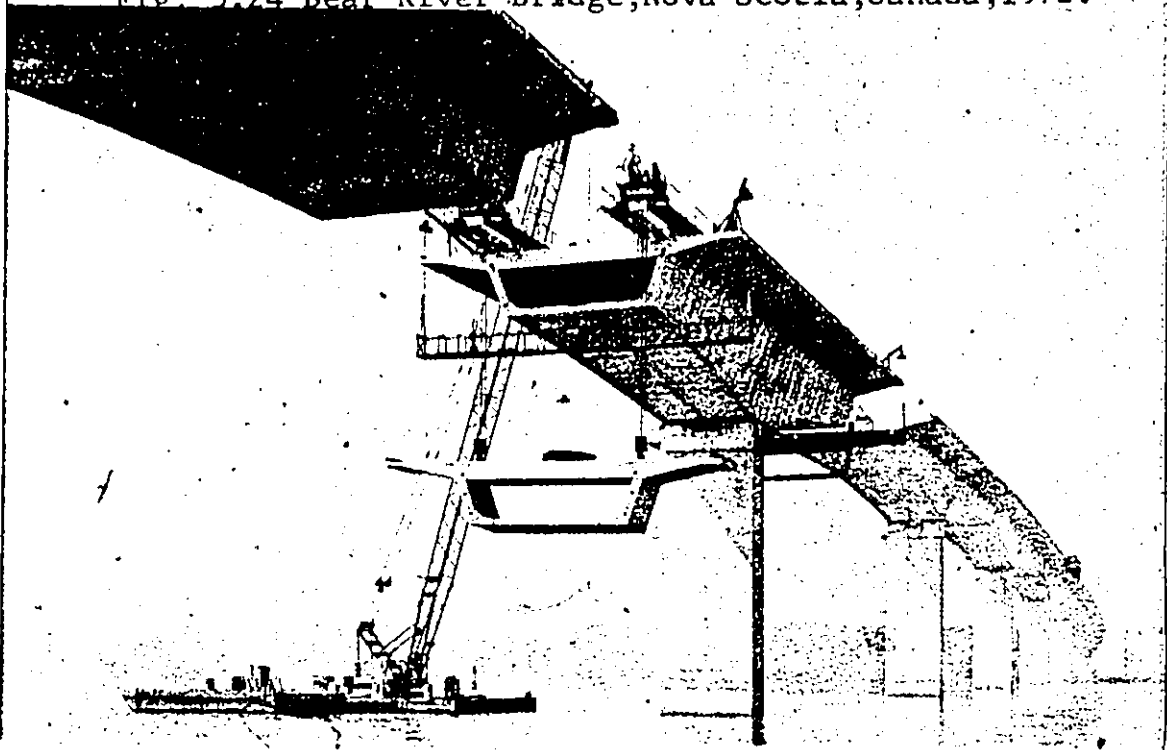


Fig. 5.25 Saint-André-de-Cubzac Bridge over the Dordogne River, France, 1974.

The second method uses a launching beam anchored on the finished deck and a moving winch carrying the erection equipment (Fig. 5.25, Ref. 5.1). This method uses a starting element as the base, usually comprised of the pier element which is either cast in place or prefabricated. In the second case it can be placed by a crane or by a special structure anchored on the top of the pier and removed once the pier element is in place (Fig. 5.26, Ref. 5.11).

The third method, used for the construction of viaduct in Tours sur le Loire, France, consists of placing the elements with a movable crane striding over the viaduct itself and moving on service roads temporarily erected on each side of the structure (Fig. 5.27, Ref. 5.12).

The fourth method, expensive at first sight, is very useful for the construction of very long bridges (Fig. 5.28, Ref. 5.9). This method will be examined in more detail in the example of Oleron Viaduct.

The placing of the segments can be separated into the following operations. Once the pier segment is installed or cast in place, the two adjoining segments are erected simultaneously or one after the other. First, the common vertical surfaces of the already placed segments and the new

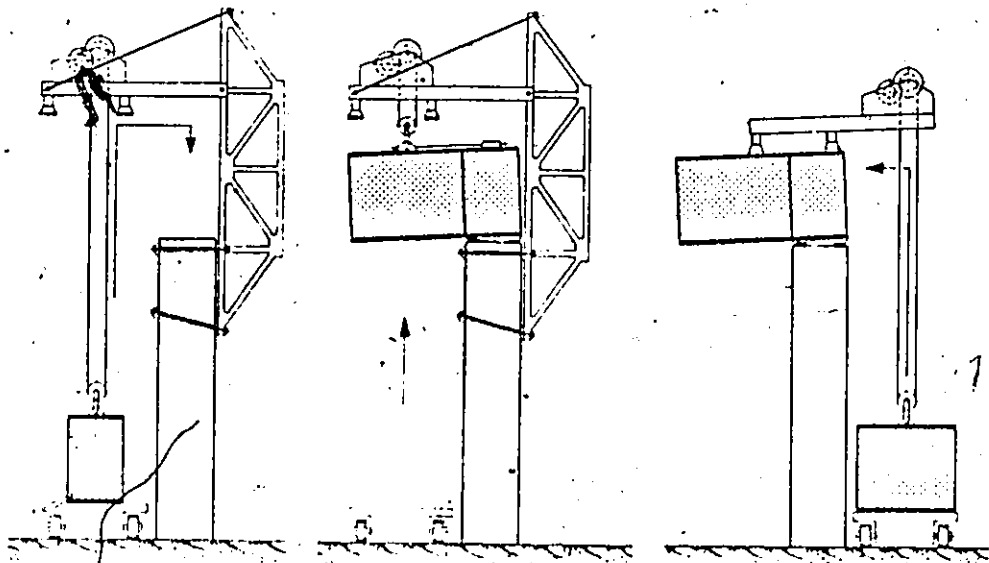


Fig. 5.26, Placing the pier elements by a special structure.

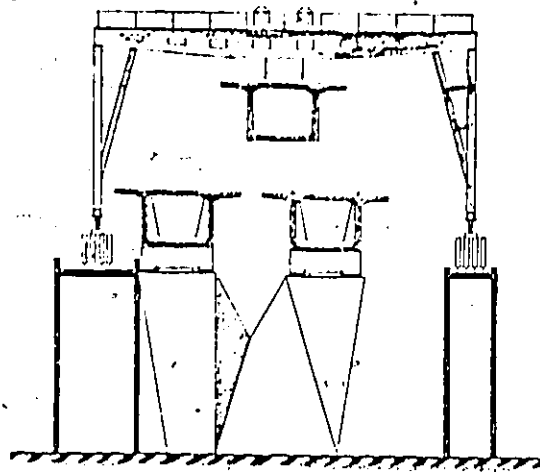


Fig. 5.27 Placing the segments at the Tours-sur-Loire Viaduct.

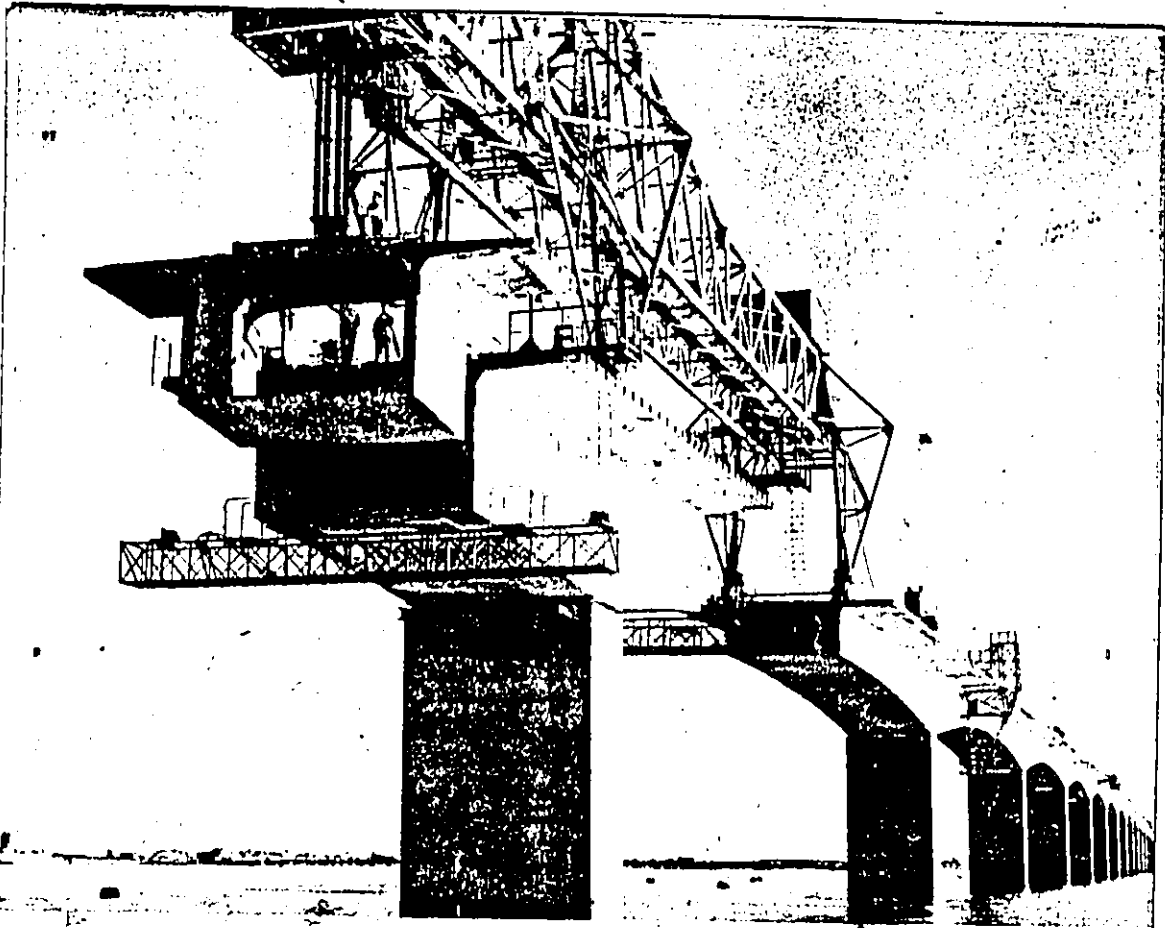


Fig. 5.28 Bridge between the Isle of Oleron and the Continent, France, 1965

one are coated with an epoxy bonding agent. Once the segment is in place and the camber checked and corrected, a temporary post-tensioning is applied to hold the segments together and squeeze the joint until the final post-tensioning is completed.

It is important that all these operations (epoxy application, erection assembling and temporary post-tensioning) be completed during the pot life of the bonding agent.

To complete the cycle the final prestressing is done and the temporary one is removed.

The prestressing steel layout is identical to those erected using cast-in-place segments. That is, all the tendons are on the top slab except for those placed in the positive moment regions which are in the bottom slab. Once the two cantilevers of the same span are completed and the bottom prestressing steel in place, the gap of approximately 3 feet is cast in place.

This method, as briefly explained, is the usual method used almost everywhere. However, a new method was used for the viaducts of the Rhône-Alpes highway in France as well as for the erection of the segments of the Olympic Stadium in Montreal.

It consists of placing the first segments near the pier on a falsework and cantilevering the remaining ones (Fig. 5.29, Ref. 5.1) using an exterior temporary prestressing (tie bars) (Fig. 5.30, Ref. 5.1) along the deck

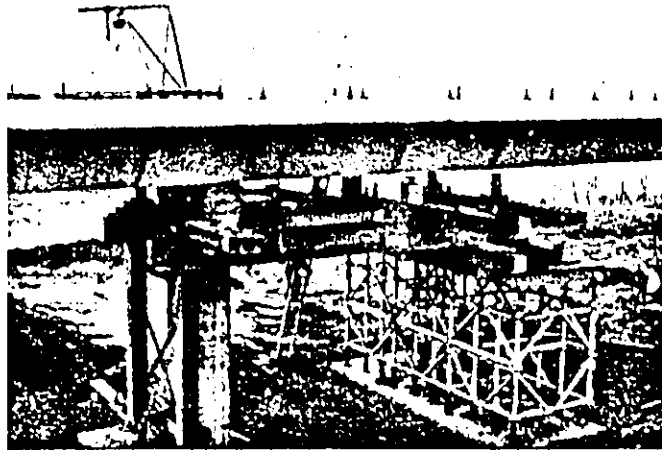


Fig. 5.29 Cantilever support.

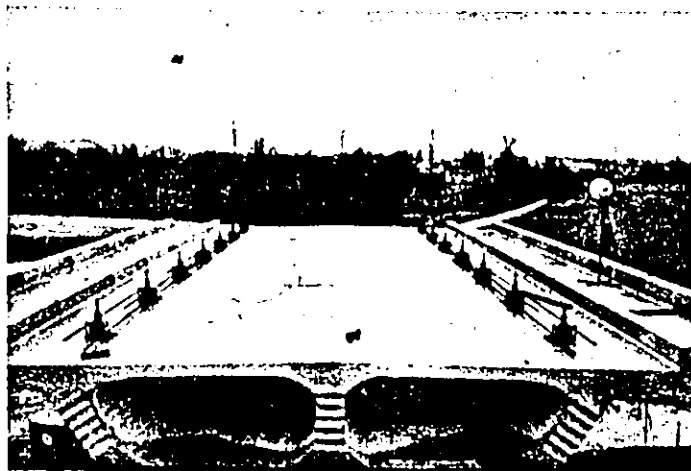


Fig. 5.30 Temporary cantilever prestress.

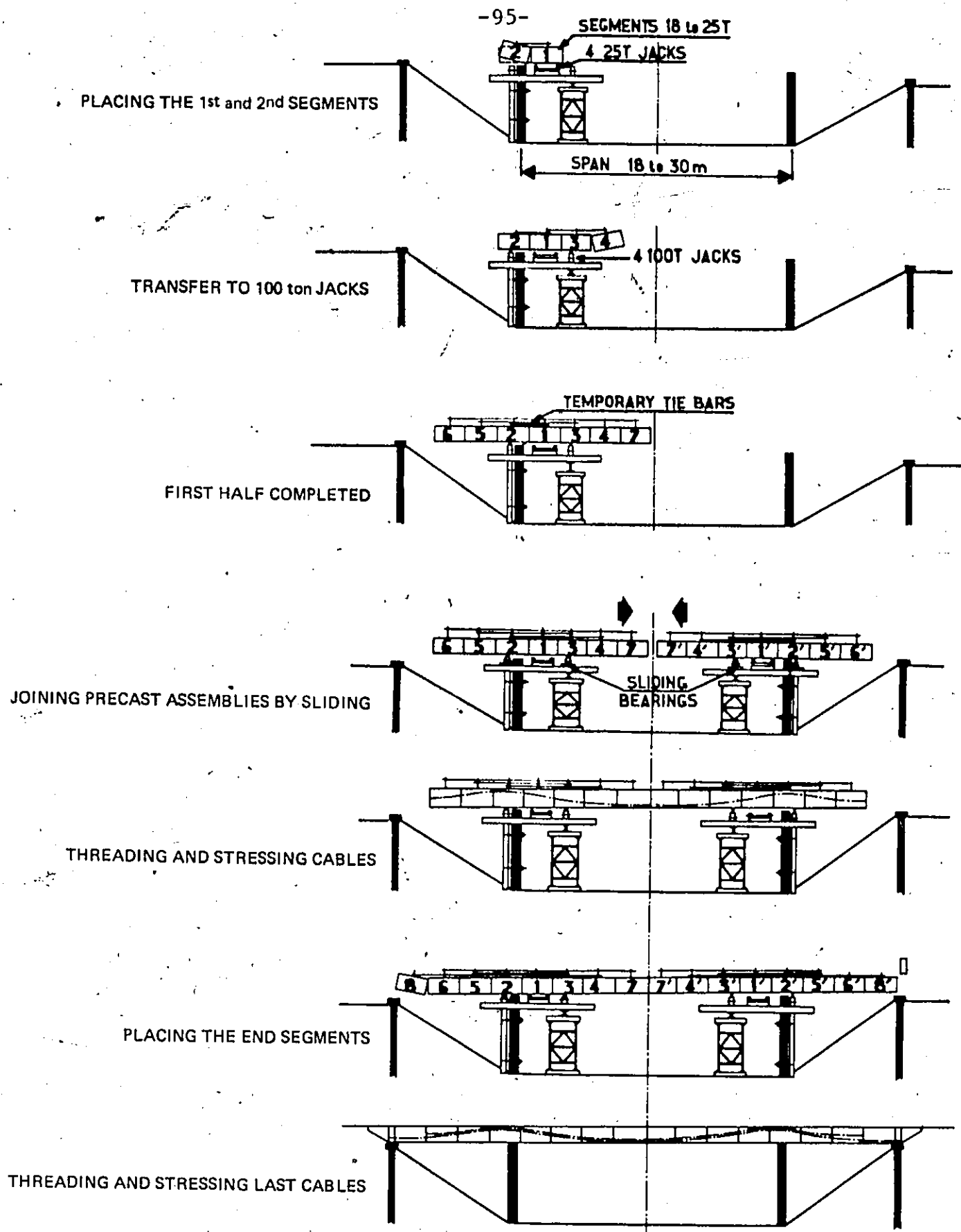


Fig. 5.3I Alpine Motorway Overbridges erection scheme..

surface. Once the complete bridge is erected, the final prestressing is placed and the exterior one removed. The difference between the two methods is that the prestressing steel layout in the latter one is very similar to that of a cast on falsework deck (Fig. 5.31, Ref. 5.1).

5.3.4 Joints and Epoxy

As already pointed out, match-cast joints for the segments are a necessity. Another special feature of this kind of structure is the use of the epoxy bonding agent between the segments as previously mentioned.

This epoxy is applied using a trowel or brush to obtain 1/16 to 3/32 inch (1.6 to 2.4 mm) thick coating on both surfaces to be joined (Ref. 5.10).

During the first years of the cantilevering method with prefabricated segments, the epoxy was used to take tensile stresses due to bending moments as well as shear forces. But as in segmental erected bridges, there is compression throughout the section or very small tensile stresses, the use of epoxy to take these stresses is unnecessary, although it is essential to have a high resistance in shear stress.

But since these agents cannot resist any stress until several hours after they have been applied, initially they are used only as a lubricant for the easy placing of the segments and to achieve a perfect joint.

To insure the stability during erection and the proper alignment of the segments, each one is provided with shear keys at the faces of the webs and at the top slab (Fig. 5.32, Ref. 5.9). The web keys prevent the vertical sliding of one segment relative to the other, while the slab keys guide and center the segments.

In order to free the lifting device and accelerate the erection, an outside temporary post-tensioning force, realized by tie-bars, is applied on the top and bottom slab. These two forces with the shear keys insure the complete stability of the erected segment until the final prestressing is applied. On Fig. 5.33 (Ref. 5.11) it can be seen that the location of the shear keys has to be chosen carefully in order that the resultant of all the forces gives a compression force all over the section of the joint (Ref. 6.1 and 6.17). According to Ref. 6.16 this compression has to be at least 30 psi until the permanent tendons are stressed.

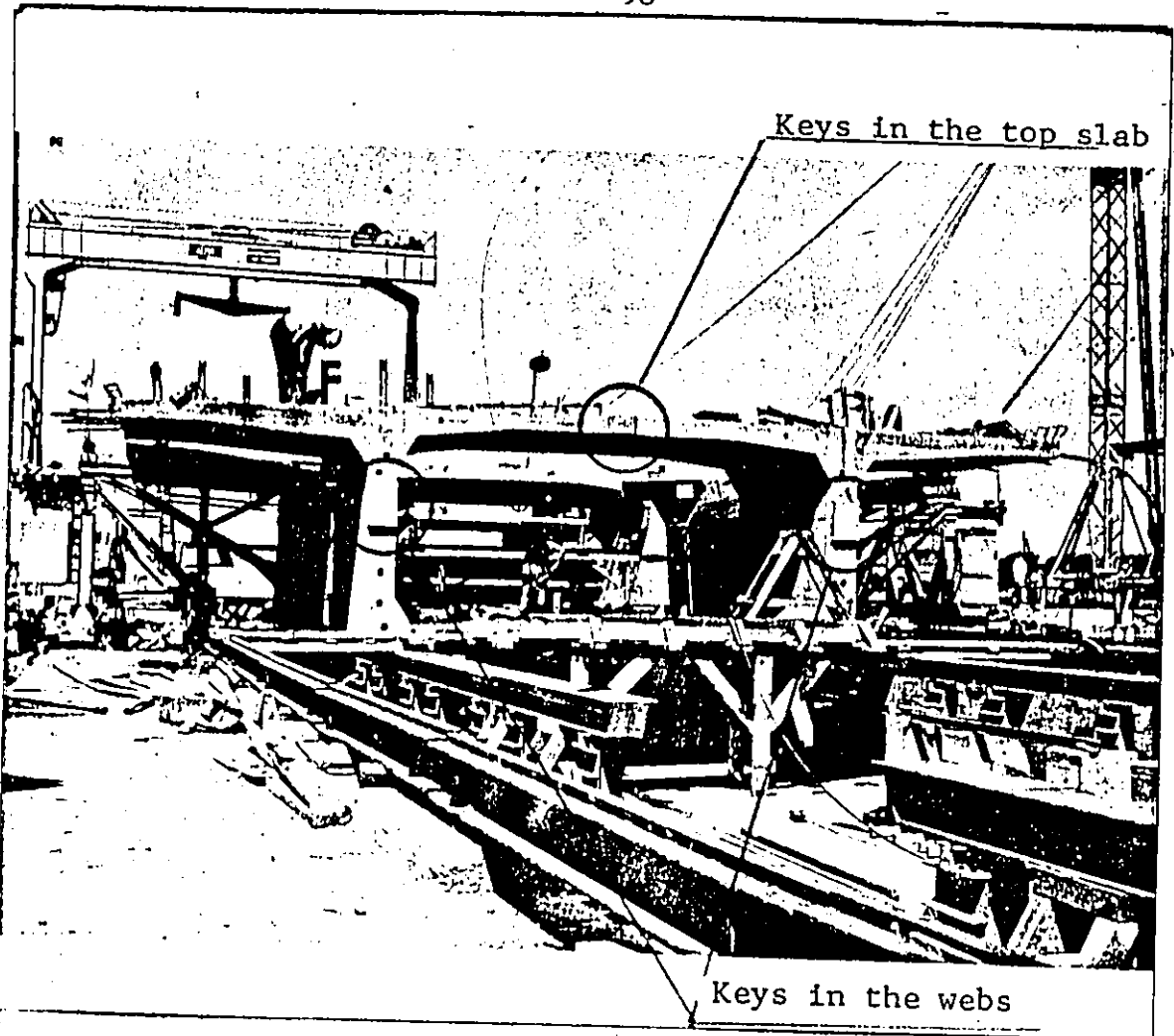


Fig. 5.32 Keys of a prefabricated segment.

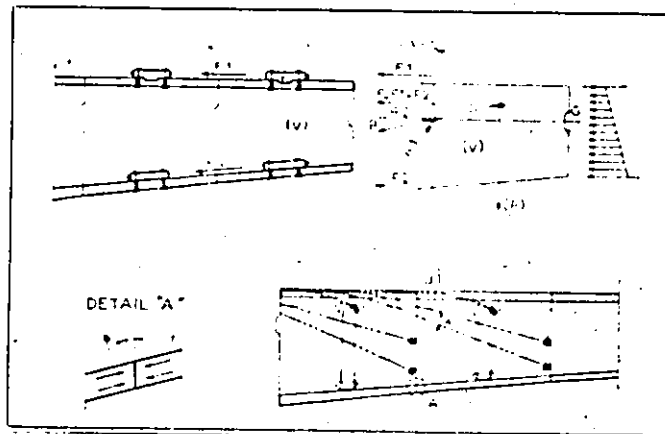


Fig. 5.33 Forces acting upon a match-casted joint.

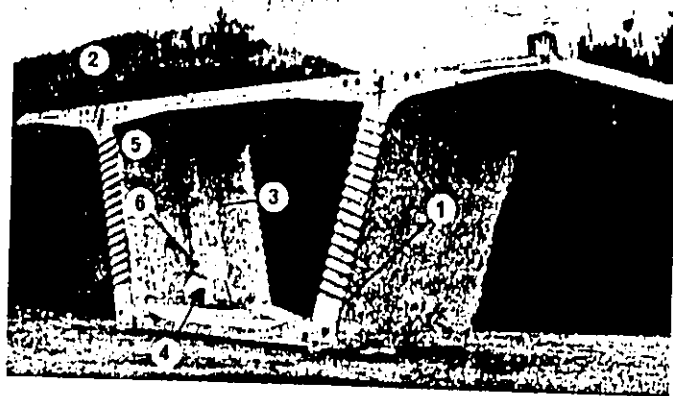


Fig. 5.34 Precast segment with multiple keys and internal stiffener. (1) Castelled web keys, (2) slab key for alignment, (3) web stiffener

After its hardening, the bonding agent will insure a water-proof joint which is very important in countries where salt is used to melt the snow and the ice during winter.

More recently, multiple keys (Fig. 5.34, Ref. 5.11) were developed which permit us to neglect the shear capacity of the epoxy which is now used only to:

- (a) lubricate the adjoining surfaces of the segments;
- (b) realize a perfect joint;
- (c) waterproof the joint.

In Ref. 6.15 there are some interesting results of tests made with the glued joints for the Rio Niteroi Bridge.

5.4 Comparison of the Two Discussed Methods

It is difficult to make any definitive comparison between the two methods, since each one has its advantages and disadvantages. Every particular case has to be the object of a detailed study taking into account the various factors such as spans, total deck surfaces, use and amortization of the equipment, possibility to cast in place,

availability and capacity of the lifting devices, encumbrance in urban areas, etc.

As a general rule, the cast-in-place requires less investment, the greatest being for the form-traveller which in some cases can even be rented from a specialized company (Grand'Mère Bridge). But on the other hand, the construction speed is slower. It is a good method for relatively shorter bridges and for bridges with long spans where the weight of the segments is too heavy to be handled by the cranes.

In the prefabricated method, the work progresses quicker, we get better concrete quality and finished product, and finally we get a better control of the deflection since almost all the concrete shrinkage has taken place prior to erection.

On the negative side, the investments are higher because a prefabrication plant is required and the handling and erection equipment is more costly.

It seems to be the method used mostly for long bridges, but it has been used successfully in urban areas and for small structures with considerable repetition where the cost amortization is possible on many units.

5.5 Deck Stability During Construction

As was mentioned in 5.2.2.1, unbalanced conditions can occur if for some reason the segment of one cantilever has to be poured prior to its symmetrical partner. But even in a symmetrical progression of the work, the deck must be securely anchored to prevent an overturning due to a sudden change of the loads between the two cantilevers. This anchorage can be either temporary or permanent.

5.5.1 Permanent Anchorages

This can be realized

- (a) With a balanced abutment (a heavy concrete box serving as counterweight) anchored to the rock or to a foundation with post-tensioning steel (Fig. 5.35, Ref. 5.2).
- (b) With box section piers having two of their walls extending into the deck webs. These piers must be flexible in order to avoid high stresses caused by the deck deformation.

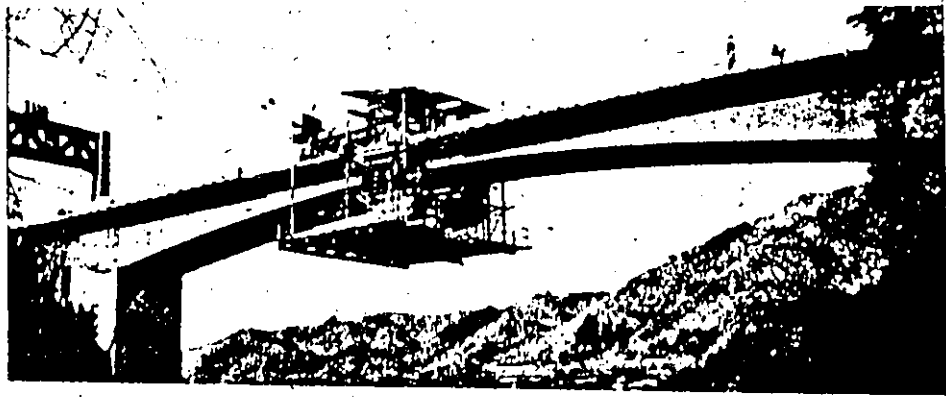


Fig. 5.35 Ranzan Bridge in Japan.

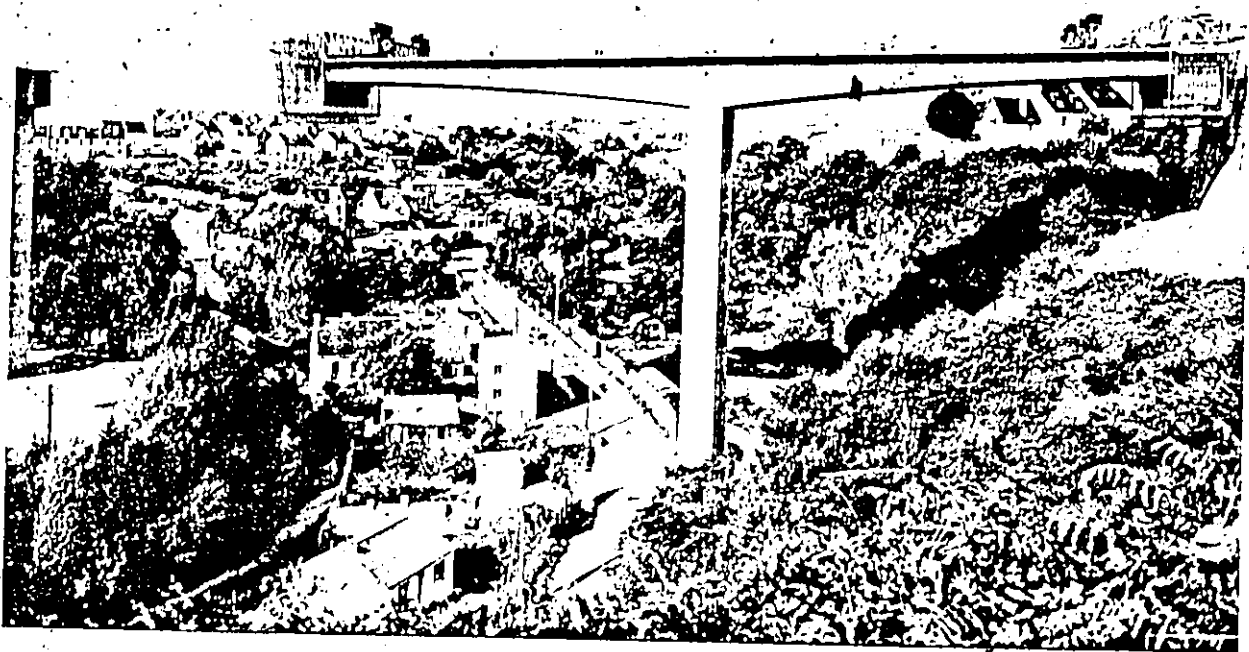


Fig. 5.36 Moulin à Poudre Bridge, France.

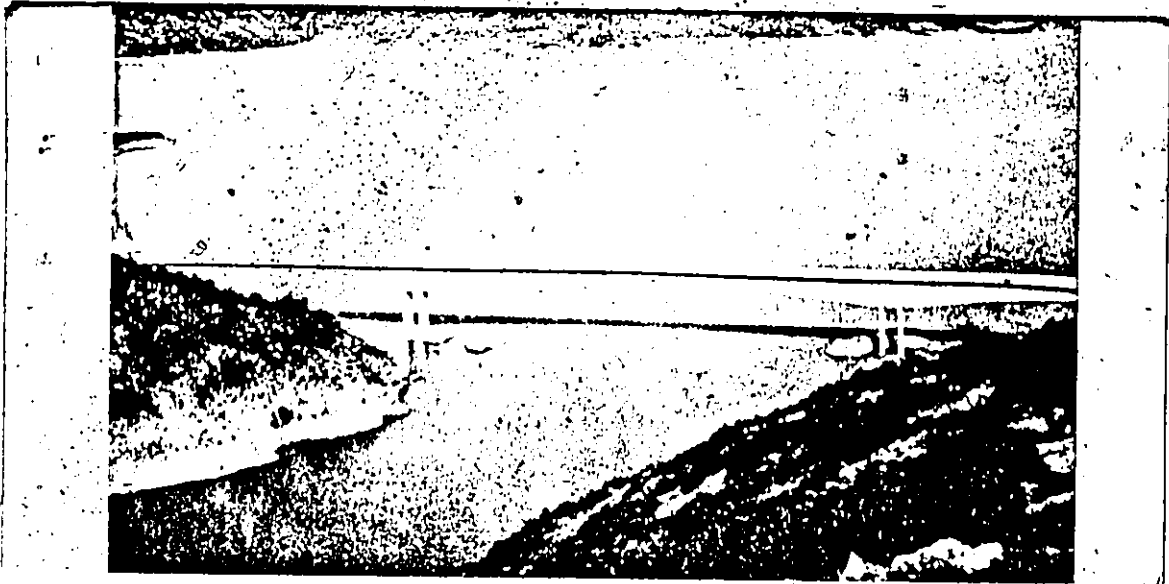


Fig. 5.37 Tatarna Bridge in Greece.

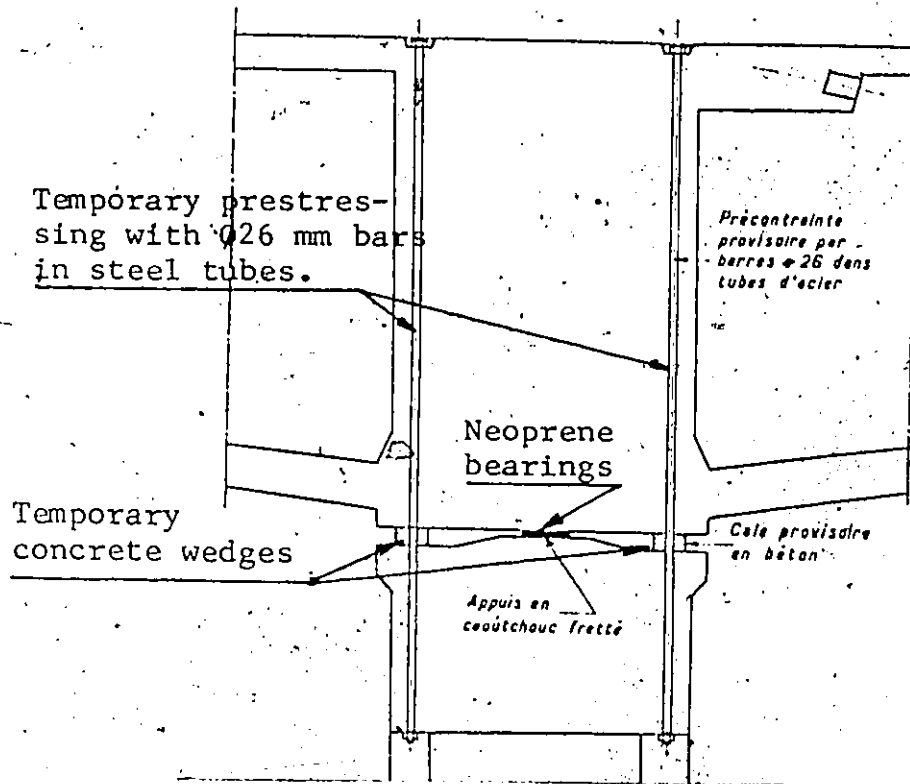


Fig. 5.38 Lacroix-Falgarde Bridge, France. Temporary anchoring of the deck to the pier.

One way to achieve that is to use very high piers (Moulin à Poudre Bridge in France, Fig. 5.36, Ref. 5.7).

- (c) With piers formed by two parallel flexible walls extending into the deck diaphragms (Chillon Viaduct, Switzerland (Fig. 4.8, Ref. 4.4), Tatarna Bridge, Greece (Fig. 5.37, Ref. 5.2)).

5.5.2. Temporary Anchorages (General Case)

Since the deck in the final stage is usually simply supported on the piers (on teflon or neoprene bearings or with Freyssinet concrete hinges), the temporary anchorage can be achieved as follows:

- (a) Anchoring the deck to the pier by vertical prestressing (Fig. 5.17).
- (b) Supporting the deck on the scaffolding used to cast the first elements on each side of the pier or on specially erected piers (Medway Bridge, England, Fig. 7.5).



5.6 Construction by the Pushing Method

5.6.1 General

The pushing method consists of prefabricating, on the fill behind the abutment and on the longitudinal axis of the bridge, the deck by long segments and consolidating them by prestressing one against the other. As the construction progresses, the completed segments are pushed on sliding bearings towards its final position (towards the first support).

It is clear that this method is completely different from those previously described. The only common thing and reason why it is included here is that the deck is cast in a series of segments and it is cantilevered out to the piers.

This method, although new in the concrete construction field, has been used for a long time in steel structures.

The deck of these steel bridges is erected in prefabricated segments on the ground and it is progressively pushed to its final position by sliding on rollers, using the bottom member as a rail.

Since the steel works as well in tension as in compression, and since the friction on the rollers is negligible due to the small weight of the deck, this kind of construction is possible for the steel bridges. But since concrete lacks both those properties, the application of the pushing method on concrete bridges was impossible until, first, the use of prestressing methods permitted the structure to withstand reversible loading conditions, and second, the development of bearings with a very low friction factor permitted the displacement and precise positioning of these heavy structures (Ref. 5.14).

The first application seems to be in 1959 for the Ager Bridge in Austria and the idea was suggested by Willie Baur of Professor Fritz Leonhardt's office.

5.6.2 Description

5.6.2.1 Casting and Advancement of the Deck

A flat surface is built behind the abutment having a length of about two segments (Fig. 5.39, Ref. 5.13).

The segments are cast in a stationary mould as for the short line described in 5.3.2.2.

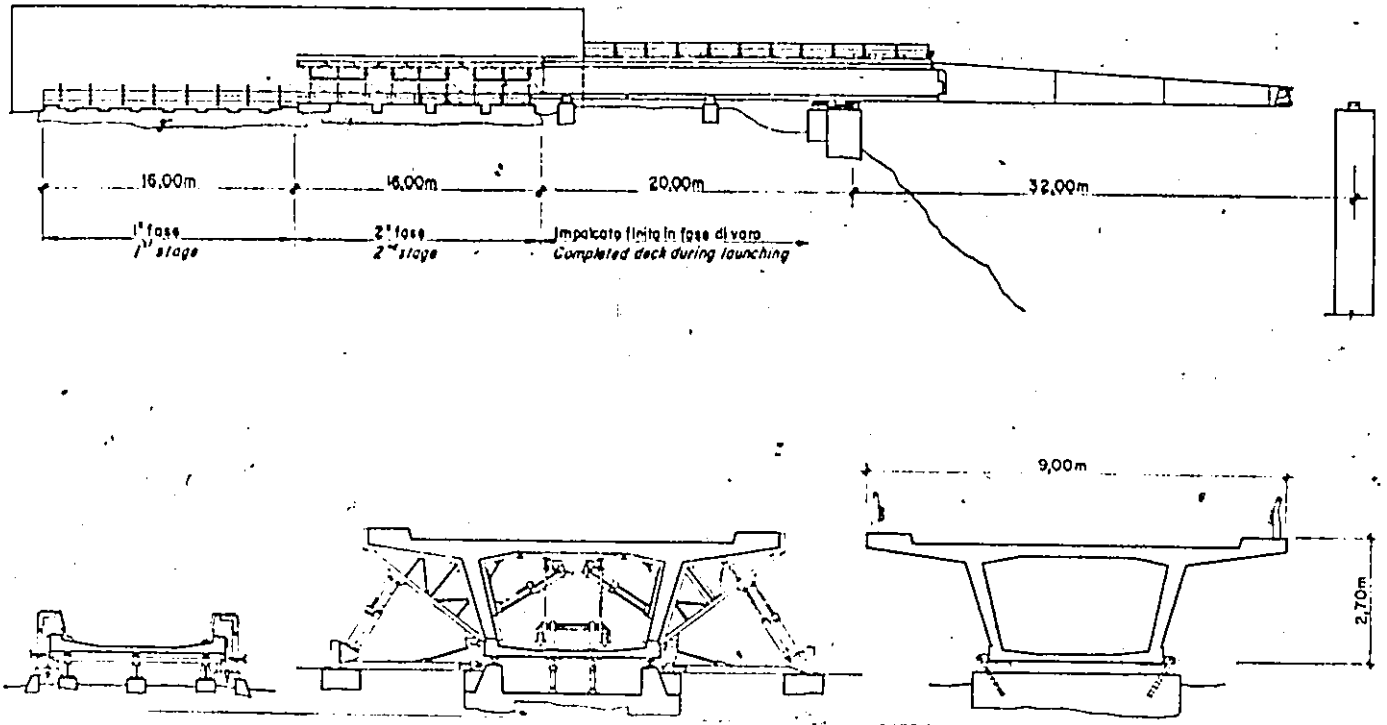


Fig.5.39. Prefabrication of the segments and pouring phases of the Val Restel Viaduct, Italy.

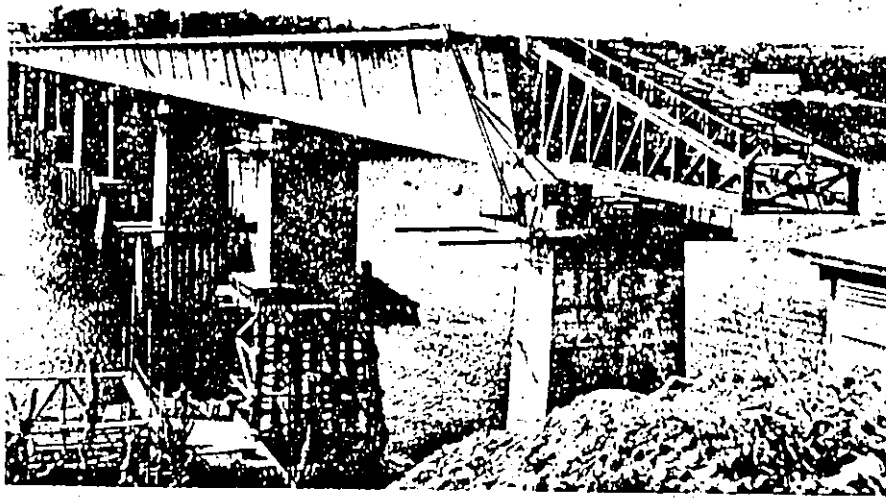


Fig.5.40. The steel nose of the bridge over the Caroni in Venezuela.

First, the bottom slab is poured on a steel form, in order to have an almost polished bottom surface, and then the webs and the top slab.

Once the first segment is finished, a steel extension nose is fixed to the front (Fig. 5.40, Ref. 5.14) and it is pushed towards the river for a distance equal to one segment length. The second one now is cast directly against the first and after the hardening of the concrete both segments are pushed towards the river. The mould is now ready to cast the third segment (Fig. 5.41). This cycle is repeated until the whole deck is cast.

The pushing is achieved using jacks which are supported on the abutment and which pull the deck through tension bars. The jack's capacity has to be calculated using a friction factor of the bearings of at least 5%.

It is clear that a particular feature of this kind of structure is the continuous variation of section forces at any point along the deck and at any time during the construction.

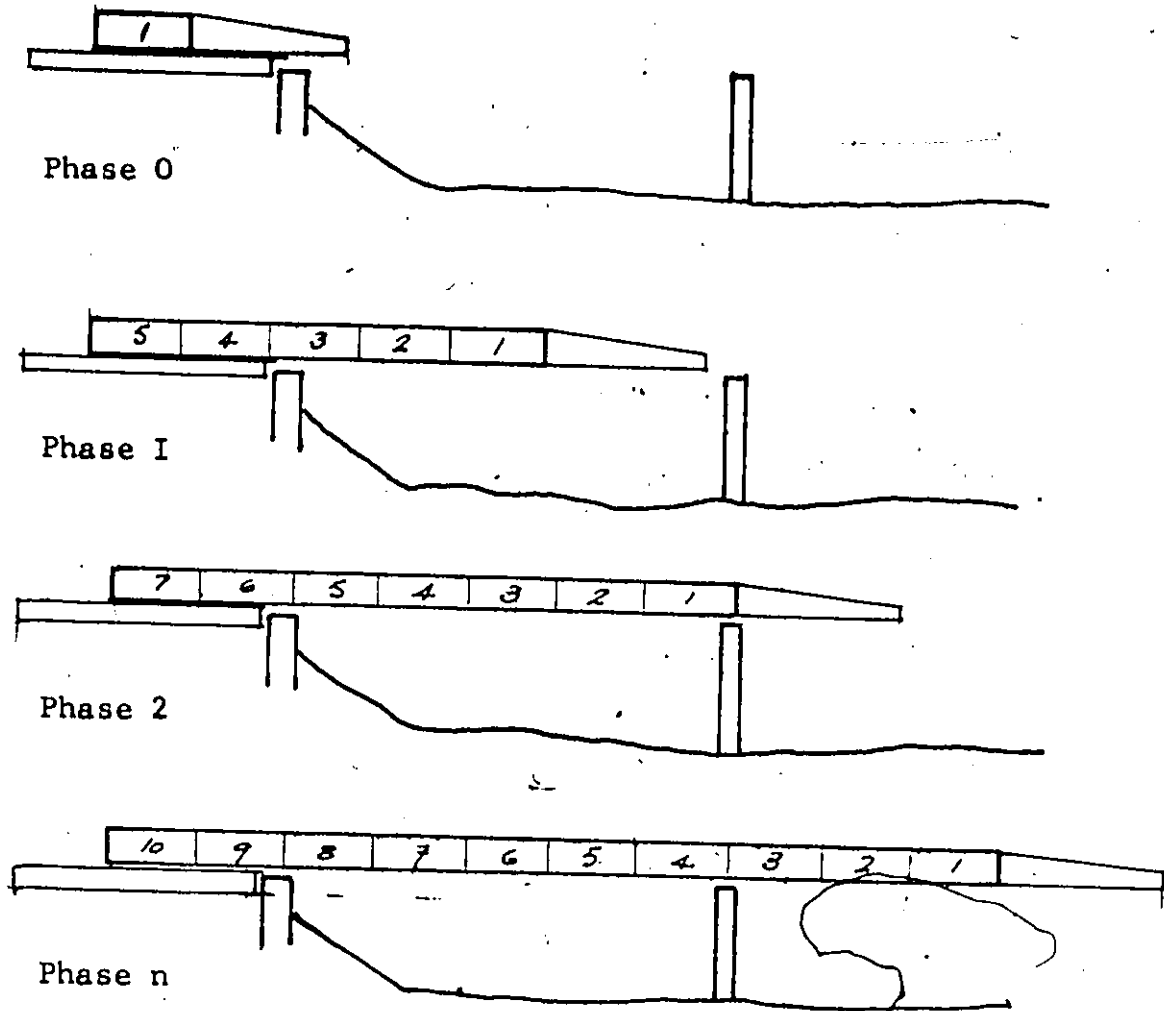


Fig. 5.4I Construction sequences of a bridge built by the pushing method.

Thus, in Fig. 5.41 at phase one, the deck is a cantilever with negative moments throughout its length. At phase two, when the steel nose rests on the first pier, the statical system becomes a simply supported beam with positive moments throughout. Subsequent steps (phase three, etc.) are simple or continuous beams with or without an overhang.

5.6.2.2 Reinforcement

The segments are reinforced longitudinally with mild steel and sometimes with an axial prestressing force created by cables placed in the top and bottom slabs (Fig. 5.42, Ref. 5.14). This prestressing can be avoided during construction either by using sections with a high depth to span ratio (12 to 15) or by reducing the portion of the deck in cantilever. In both cases the bending moments can be taken by mild steel only even if that causes some small tensile stresses or hairline cracks. These will disappear once the final prestressing will be applied.

The reduction of the cantilever length can be realized by using temporary piers, or by using a longer

steel nose, say, about 60% of the span.

At Luc Viaduct in France, a steel mast with temporary stays (Fig. 5.43, Ref. 5.16) was used behind the nose to reduce the section forces in the cantilever.

These masts rested on the webs of the deck. Using hydraulic jacks, the tensile force in the stays can be adjusted for each stage. For each step, the stresses in the deck are calculated and the necessary force is applied to the jacks in order to keep these stresses in the permissible range.

Once the deck is in its final position, the longitudinal prestressing is placed. This can be of two kinds (Fig. 5.42):

- (a) Concentrated cables inside the box adjacent to the webs.
- (b) Cables placed in the ducts provided during the casting in the webs.

The choice depends on the required magnitude of the prestressing force.

Straight cables for the axial prestressing.

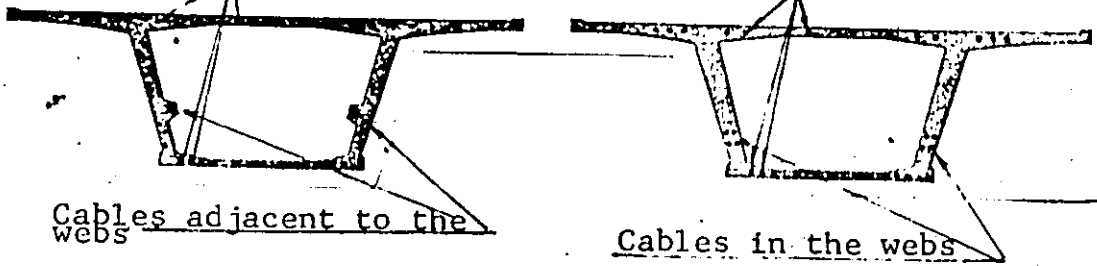


Fig. 5.42 Two methods of placing the longitudinal pres. steel

viaduc du Luc
Luc viaduct

Placing of the steel nose. Pouring and pushing of the first span.

Pouring of the second segment. Placing of the steel mast and pushing of the first two segments.

Pouring of the third segment. Pushing of the first three segments.

Pouring of the fourth segment. Pushing of the first four segments.

End of the pushing. Removal of the steel nose and mast. Placing of the final bearings. Final prestressing.

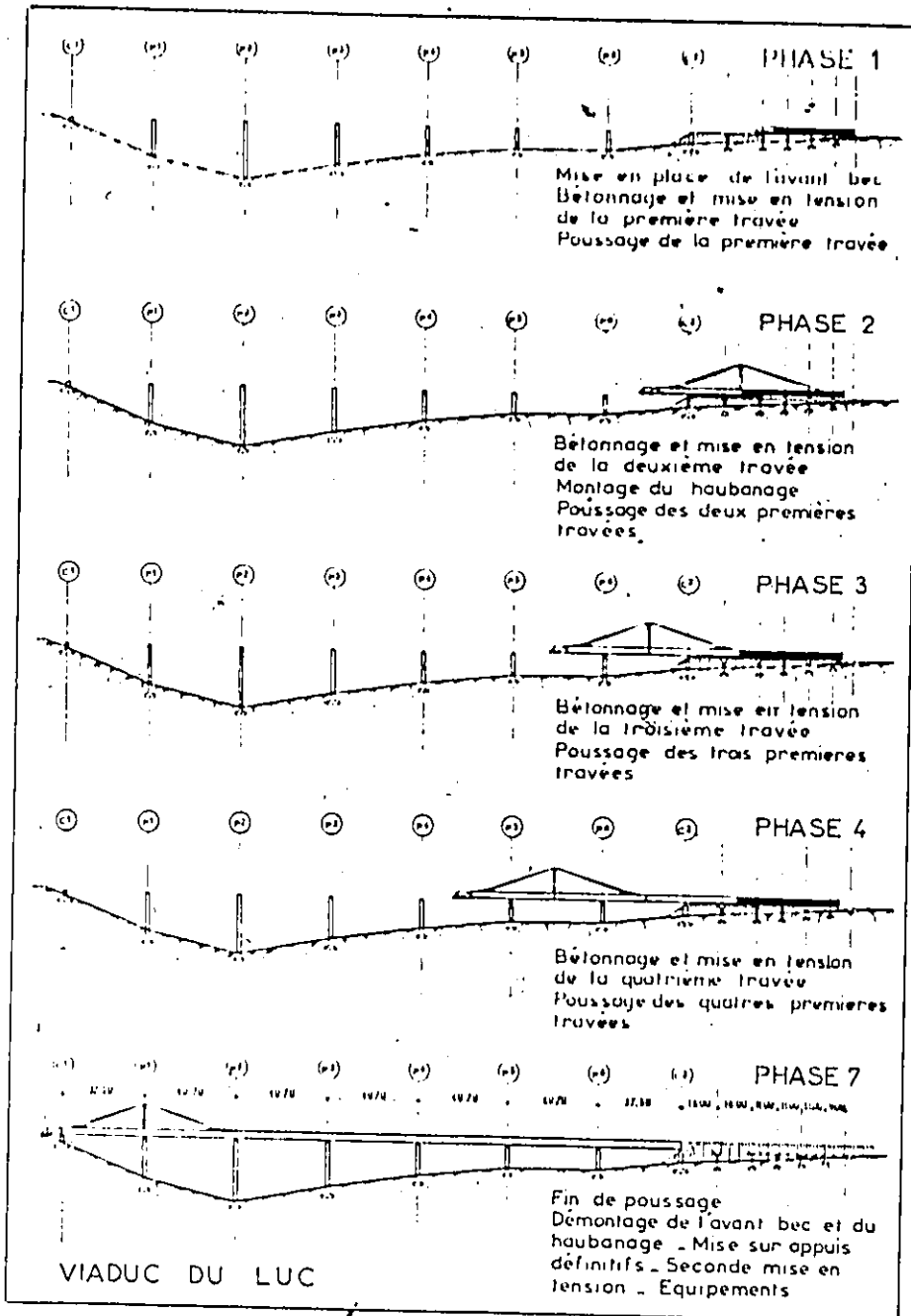


Fig. 5.43 Construction phases of the Luc Viaduct, France.

If during the pushing stage the longitudinal post-tensioning was not used, then the final post-tensioning will be high and it will be more convenient to place it outside the webs.

On the contrary, if there was some post-tensioning, additional requirements will be small and it can be placed in the webs.

5.6.2.3 Sliding Apparatus

A sliding device is placed over each pier and under each web to allow the movement of the deck.

The first devices were made with neoprene and teflon fixed by friction to the bottom face of the deck and sliding on a stainless steel plate fixed to the top of the pier. After the deck was moved about 3 feet (1 m) the whole structure was lifted on jacks (Fig. 5.44, Ref. 5.17) and the sliding devices returned to their original position. The cycle was repeated after the deck was lowered. The drawbacks of this system are evident.

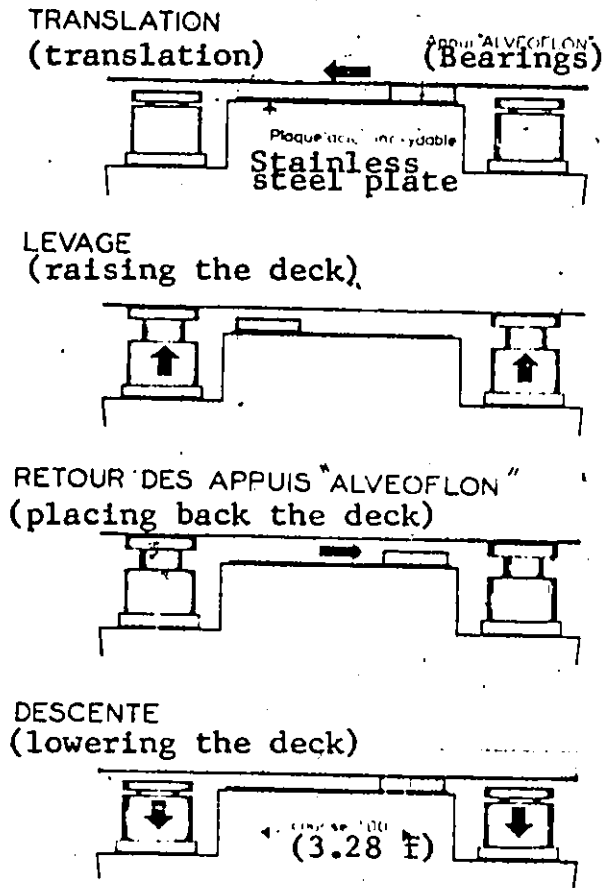


Fig. 5.44 Old principle of moving the deck.

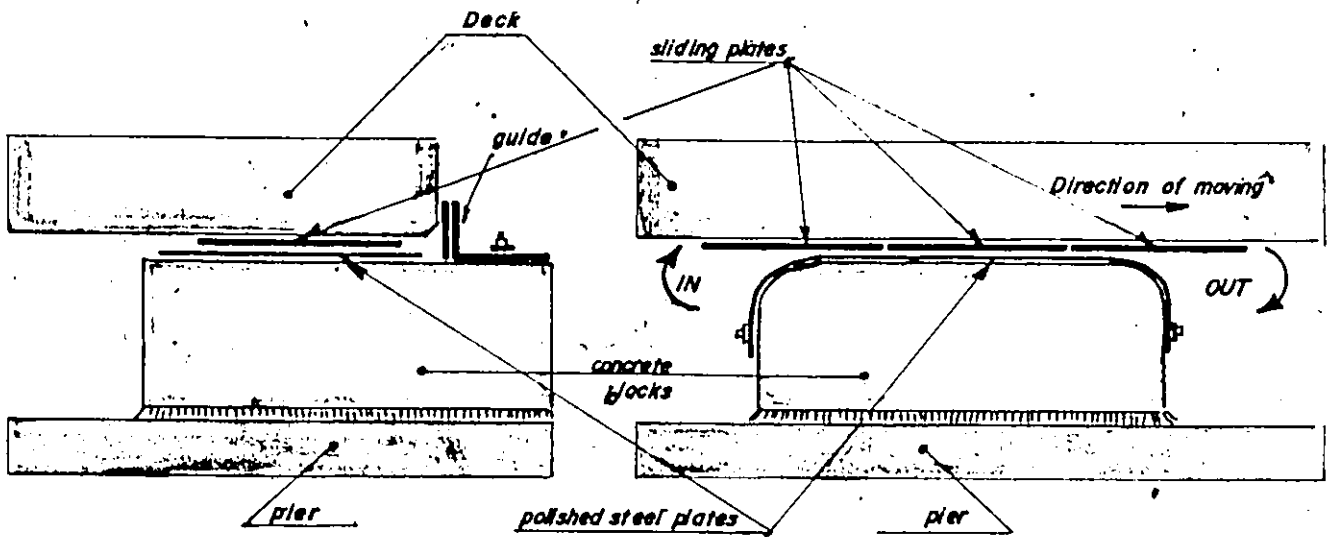


Fig. 5.45 New sliding apparatus.

Today, the device consists of sliding plates 12x12 or 16x16 inches (30x30cm or 40x40cm) made of neoprene, teflon and steel sheets placed between the deck (under each web) and a polished steel plate placed on top of the pier. These sliding plates, carried by the deck movement, are recovered at the end of the course and replaced immediately at the front of it (Fig. 5.45, Ref. 5.14). The movement speed can be 67 to 100 feet (20 to 30 m) a day. In addition, a location must be provided over each pier in order to install the jacks which will serve to lift the deck once the construction is finished so that the sliding bearings can be replaced with the final bearings.

5.6.3 Applications

The use of the pushing method is governed by the following conditions.

- (a) Deck with constant height.
- (b) Constant curve for the horizontal and vertical profiles (Fig. 5.46, Ref. 5.13).

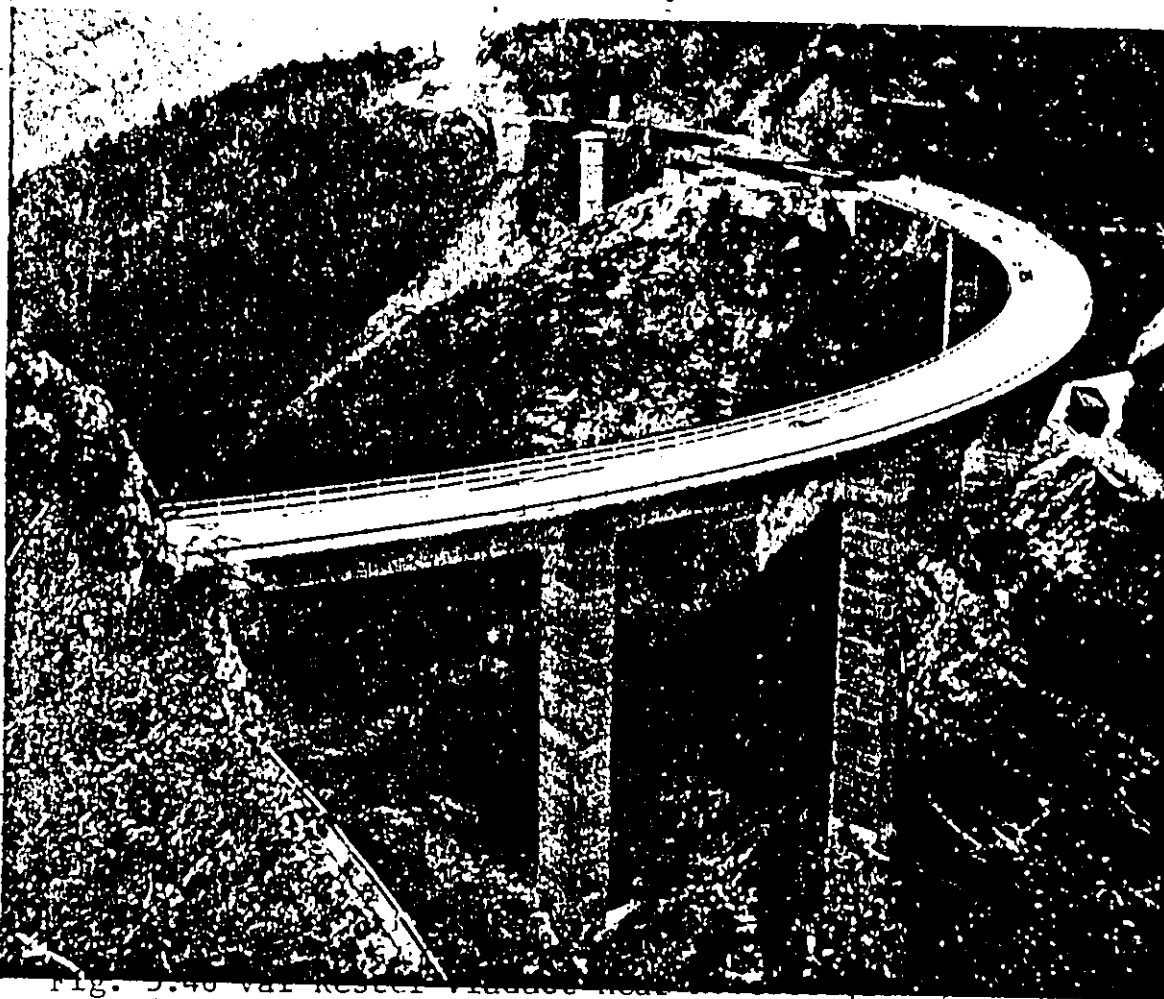


Fig. 2.40 Val Reston Viaduct near ...

This viaduct of a total length of 1050 feet (320 m) has a horizontal curve of 493 feet (150 m) and a vertical one of 8867 feet (2700 m).

- (c) Almost polished surfaces of the bottom face of the webs.

According to Ref. 5.14, this method can be used for total lengths of 490 feet (150 m) and spans greater than 100 feet (30 m) but many constructed bridges have spans of over 330 feet (100 m).

5.6.4 Advantages of the System

- (a) It combines the advantages of the prefabrication method because the whole deck is cast in a completely covered stationary mould with those of the cast-in-place method, because of a jointless structure with all the mild steel running throughout the deck.
- (b) It eliminates the need for scaffolding which is important when their use is difficult or impossible (urban areas, bridges over railways, etc.) and it reduces the required working area to a small one behind the abutments and around the piers.

- (c) The construction is done at ground level, thus eliminating any danger of accident, and the crew is working in security and comfort (Fig. 5.47, Ref. 5.13).
- (d) The initial investments are very low. The only required materials are those needed for the pushing; that is, pulling jacks, bearing plates and, if necessary, a mast, temporary piers and a steel nose.
- (e) Easy access on the entire deck and so a better control of the structure.
- (f) Rapidity of execution.

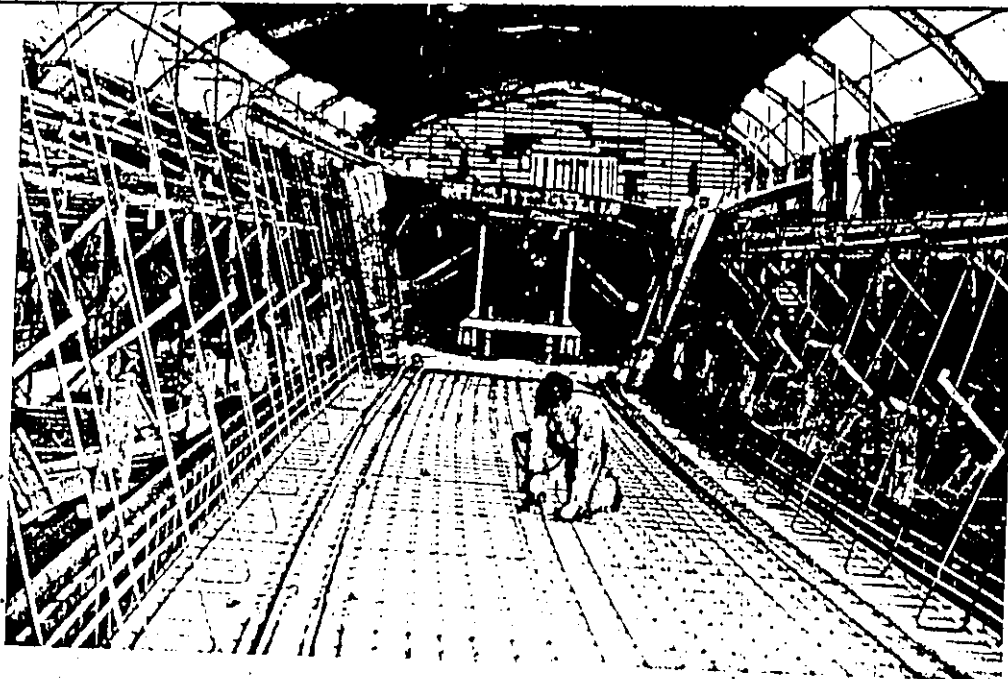


Fig. 5.47. Val Restel Viaduct, Italy.

Casting the segments in a completely sheltered cell.

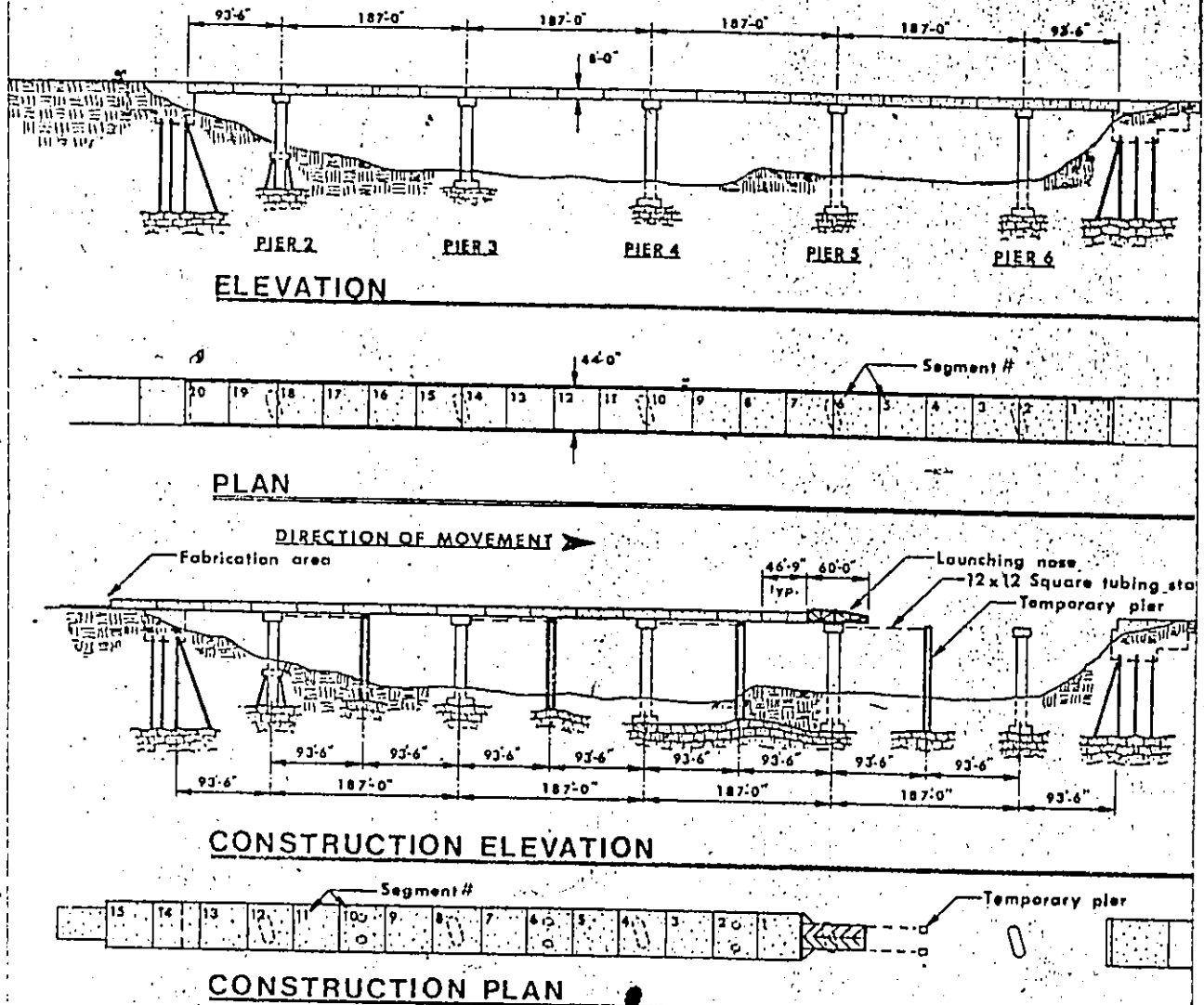


Fig 5.48. Wabash River Bridge Covington, Indiana. First incrementally launched post-tensioned box girder bridge to be built in the United States.

CHAPTER 6

DIMENSIONING AND DESIGN

As previously discussed, it is clear that just as interesting and simple as this method can be for the contractor, it is equally as difficult in concept and design for the project engineer.

In fact, although a cast-on scaffolding bridge has to be designed for the final condition only, a bridge erected by the cantilever method has to be designed for all the construction stages as well as the final one.

Hereafter will be briefly examined the criteria for the dimensioning of a bridge, and the design and checks required for its realization.

6.1 Dimensioning

For the dimensioning of a bridge constructed by the cantilevering method one must keep in mind that during the whole construction period, and sometimes after, as for the bridges with hinges in each span, the statical system is a series of cantilevers. This necessitates the placing

of the steel in the top slab and having a thick bottom slab to resist the high negative moments created during the erection.

Furthermore, if the two cantilevers of the same hammerhead are not self-balanced, bending moments will be transferred to the pier and foundation.

6.1.1 Choice of the Spans

Although this choice very often is imposed by the site conditions, one must not forget that the cantilevers have to be balanced and that the end span has to be at least equal to 70% of the next one (Fig. 6.1, Ref. 6.17) in order to avoid any uplift at the exterior bearing.

If the end span is too short an uplift is produced and the deck has to be anchored to the abutments (Fig. 6.2, Ref. 6.17) or the end span filled with the necessary ballast as in Grand'Mère Bridge (Fig. 6.3).

6.1.2 Substructure

The substructure of a bridge erected by the cantilevering method, contrary to the one cast on scaffolding,

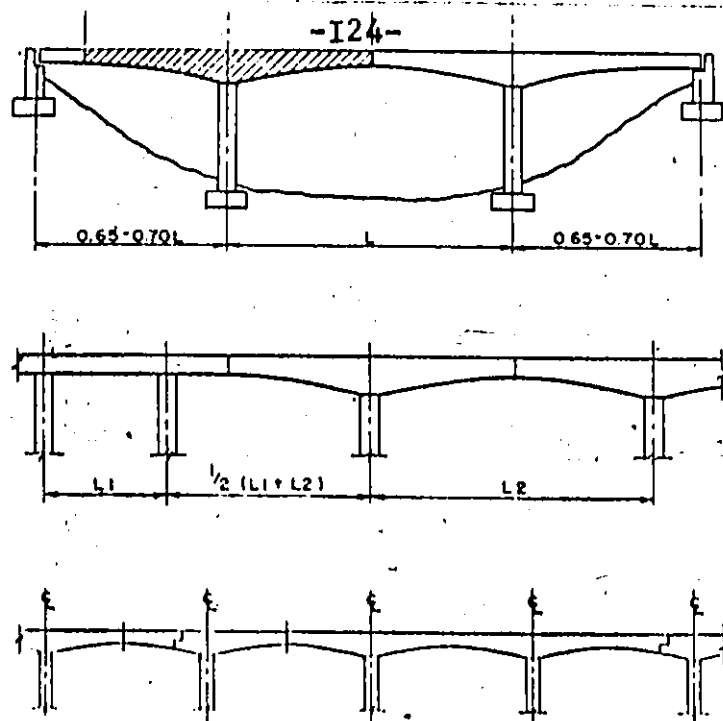
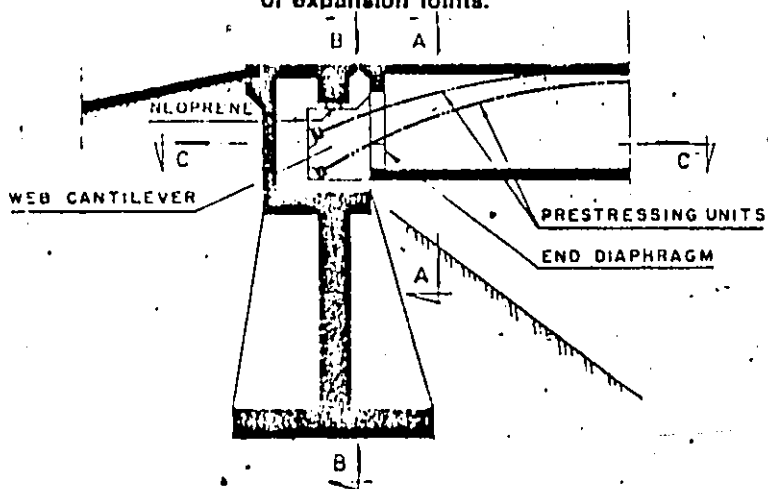
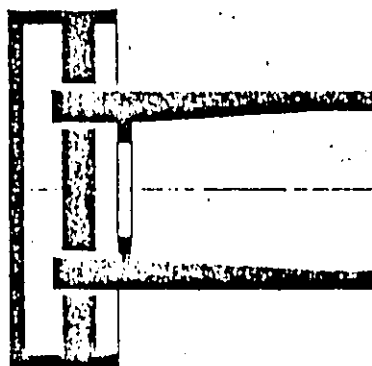


Fig. 6J. Cantilever construction showing choice of span lengths and location of expansion joints.



LONGITUDINAL SECTION



SECTION C-C

Fig. 62 End span anchor in abutment for uplift.

PREVIOUSLY COPYRIGHTED MATERIAL,
GRAPH ON LEAF 125,
NOT MICROFILMED

COPYRIGHT: GOUVERNEMENT DU QUEBEC
MINISTÈRE DES TRANSPORTS
DIRECTION GÉNÉRALE DU GENIE

GRAPH OF GRAND MERE BRIDGE

is loaded immediately after the form-traveller is on the top of the pier or after the erection of the first segment.

Thus, in addition to the vertical reaction of the deck, the piers and consequently the foundations have to resist bending moments created by the non-simultaneous erection or the lack of symmetry between the cantilevers (Fig. 6.4, Ref. 6.2), without neglecting the torsional moments due to wind loads on the superstructure (Ref. 6.16).

One method used to stabilize the cantilevers during erection is to anchor them to the pier permanently or temporarily. In the first case, the piers must be very high and flexible or the spans small enough to minimize the effects of the longitudinal movement of the deck. The best section for flexible high piers is the box or the double T (I), although the latter section presents the drawback of having poor torsional rigidity.

Long bridges are usually placed on teflon or neoprene bearings due to the longitudinal movement. But since these bearings do not provide system stability during erection, an anchorage of the deck to the piers or the foundation or even the erection of temporary scaffolding on each side of the pier may be necessary.

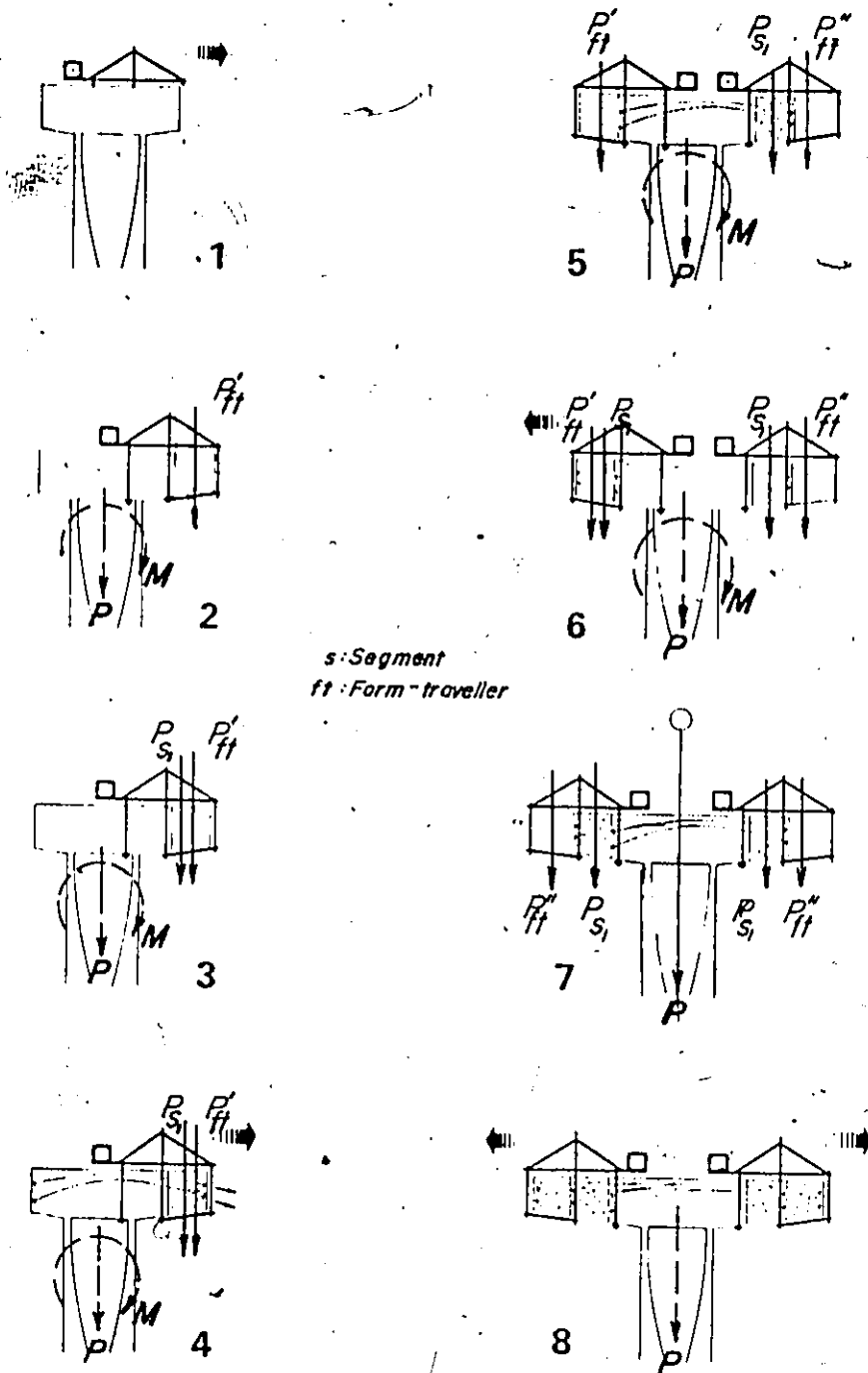


Fig. 6.4 Pier stressed by bending moments due to the non-simultaneous erection of the cantilevers

An improvement is the use of shell-piers composed of two parallel thin walls (perpendicular to the longitudinal axis of the bridge) anchored or hinged top and bottom in the deck and the foundation. (Fig. 6.5, Ref. 6.3).

This system, beyond securing the anchorage of the deck and the stability of the hammerhead during the erection, allows, due to its high longitudinal flexibility, the free movement of the bridge without creating high stresses in the substructure. In addition, in the case of walls hinged at the top and bottom, the bending moments created by horizontal loads in the deck can be completely eliminated by inclining the two walls of the pier. The reader can find a detailed study on these kinds of piers adapted to the cantilever method in Ref. 6.5.

6.1.3 Depth and Moment of Inertia of the Girder

The moment of inertia is also dictated by the fact that the structure being a cantilever, large moments are created at each end of the span and become zero at the centre. So these bridges need a high moment of inertia over the pier and theoretically a zero moment of inertia at the ends of

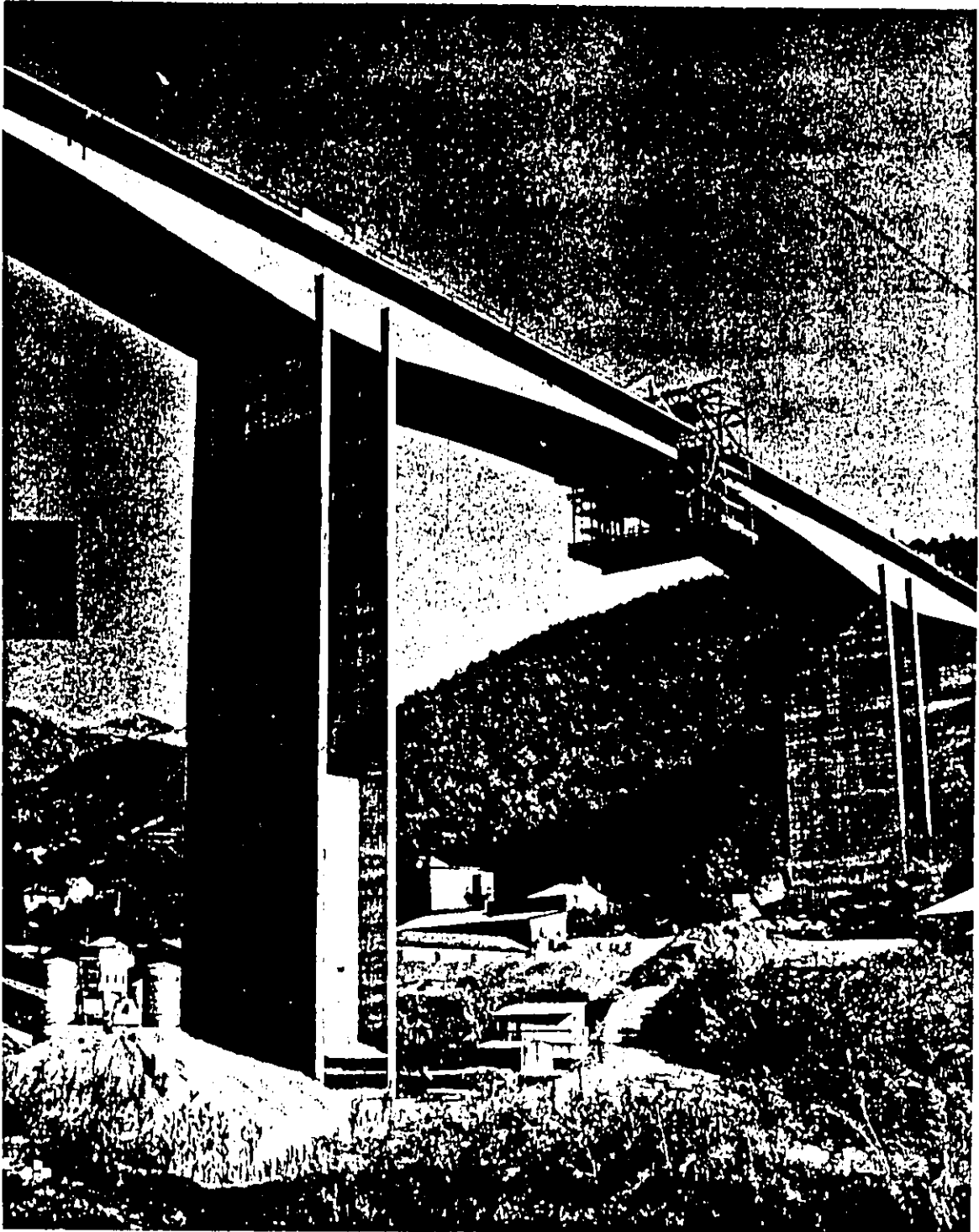


Fig. 6.5 Viaduct over the Lerone stream, Genove-Savona Motorway, Italy

the hammerheads for the spans with a hinge at the centre.

The deck height can be constant or variable. In the first case, the moment of inertia can be variable, if necessary, by changing the thickness of the bottom slab.

The deck with constant height enhances the general appearance of the structure, simplifies and facilitates the formwork and the placing of the steel, especially for the prefabricated segments. However, it requires more negative and positive post-tensioning steel, more stirrups and is extremely difficult if not impossible to use over certain spans. In such cases the use of sections of variable depth is necessary.

In Europe (Ref. 6.6) this limit seems to be 200 feet. In the United States the bridge over the Pine Valley Creek has a span of 450 feet (Fig. 6.6, Ref. 6.4) and a constant height of 19 feet 10 inches (6.04 m) but with a variable moment of inertia, its bottom slab varying between 10 and 78 inches (25.4 and 198 cm) thickness with a depth to span ratio near 1.24.

The advantages of a haunched girder are many.

- (a) The parabolic girder helps to reduce dead load moments.

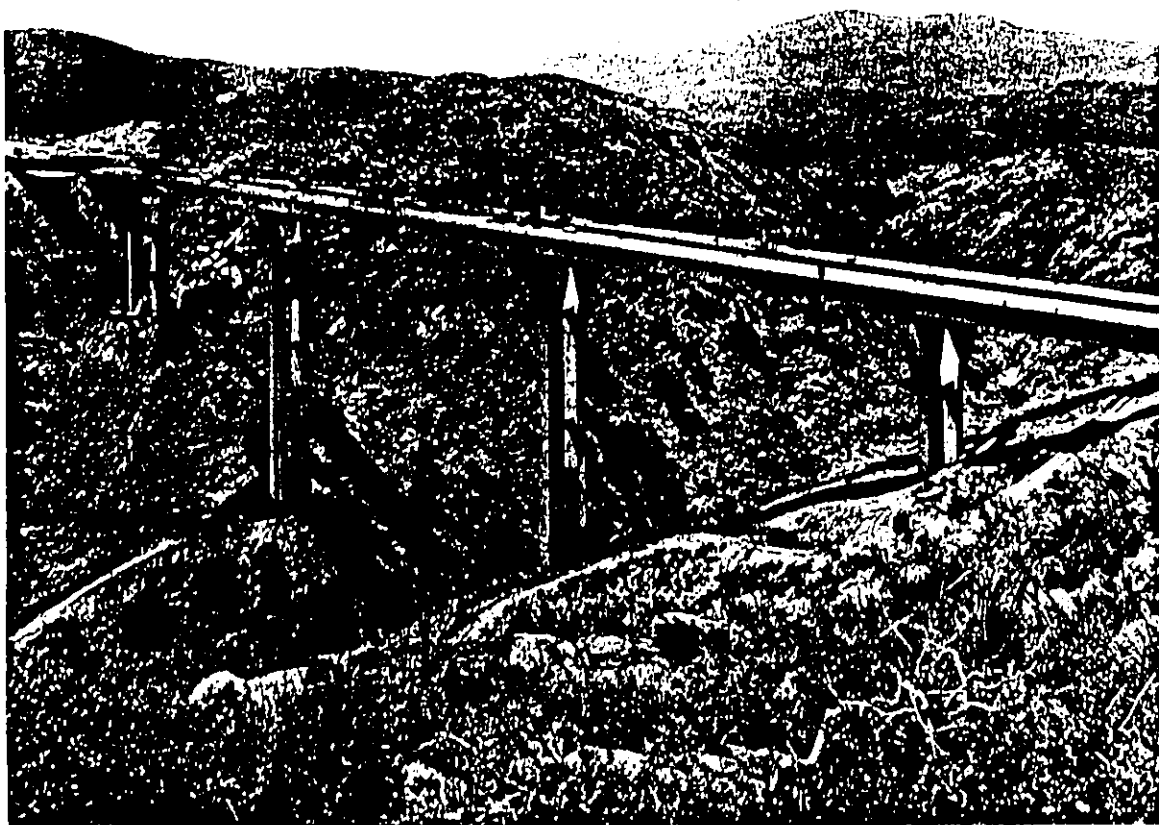


Fig. 6.6 Pine Valley Creek Bridge, California, USA.

(b) The parabolic girder is deeper in the regions where the maximum moments occur during construction and in the final structure.

(c) The shear unit stresses in a haunched girder are approximately constant over the entire span. Therefore, the web reinforcement requirements will be constant over the whole bridge.

For haunched bridges, the depth over the pier is usually $1/18$ to $1/24$ of the clear span with $1/20$ the most common value.

In the middle of the span, where the bending moments are very small, the height can be small. According to Ref. 6.7, this height can be $1/40$ to $1/60$ of the span.

Grand'Mère Bridge, with a span of 595 feet (181.3 m) center to center of bearings, has 1:62.6 at the center and 1:18.5 over the pier.

The table in Fig. 6.7 taken from Ref. 6.7 gives the dimensions of bridges erected by the cantilever method in various places in the world.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Pont	Portées L (m)	h_0 (m)	h_1 (m)	$\frac{L^2}{8h_1}$ (m)	$\frac{L}{h_1}$	Entraxes Ames (m)	Epaisseur Ames (m)
Höchst	130	3,25	7,25	292	17,9	3,25	0,30
Worms	114,2	2,47	6,50	252	17,8	1,65/4,05	0,35
Pont Dischinger	94	1,42	3,55	311	26,5	4,40/4,65	0,60
Pont Rohrdamm	72,8	1,28	3,20	207	22,8	2,80	0,40
Coblence	109,5	2,50	7	215	16,2	4,15	0,36
Tjörn	94	1,6	6	183	16,8	6,70	0,45
Kullö sund	107	1,6	7,6	194	14,5	5,15	0,45
Savines	77	1,15	4,26	172	18	5,60	0,40
Beaulieu	81,20	1,24	4,11	200	21	2,45	0,30
Villemur	52	1,00	2,90	116	17,6	4,30	0,30
	120	2	8	225	15		
	98	2,10	5,20	230	18,8		
Libourne	55	1,25	3,41	110,5	16,2	2,92	0,25
Goncellin	57,10	1,30	3,17	128	17,2	4,00	0,25
Medway	150	2,24	10,80	260	13,8	3,76	0,23
Cholsy	55	2,30	2,30	164	24	3,40	0,26
Bendorf	208	4,40	10,40	520	20,8	6,83	0,30/0,37
La Madeleine	74	1,50	3,90	175	19	4,65	0,35
Verberie	80,5	1,60	4,30	190	19,2	4,80	0,35
Rio Tocantins	140	3,40	7,70	317	18,2	6,15	0,25/0,55
Croix-l'Algerade	60,5	1,35	3,20	143	18,0	4,75	0,25
Culaba	65	1,00	3,00	175	22,5	3,05	0,25
Moulin à poudre	81,8	2,00	4,60	181	17,8	2,76	0,24
Ulua	120	1,80	5,90	305	20	2,60	0,18/0,24
Parana	109,5	2,50	5,60	267	19,5	2,95	0,40
Port-de-Bouc	43	1,28	2,58	89	16,7	5,00	0,26
La Sorges	82	1,80	4,40	191	18,7	3,95	0,35
Flevieu	55,7	2,40	2,40	160	23,2	3,15	0,35
Chooz	59	1,81	3,00	145	19,7	3,75	0,25
Khartoum	88,5	1,60	5,00	196	17,8	4,65	0,35
Pierre Bénite	84	3,60	3,60	245	23	4,50	0,25
Bordeaux	77	3,20	4,00	185	19	4,50	0,25
Oléron	79	2,50	4,50	173	17,5	5,20	0,30

Fig. 6.7 Prestressed Concrete Bridges erected by the Cantilever Method.

- Column (1) Name of the Bridge.
- Column (2) Span (L) in meters.
- Column (3) Height of the Deck over the supports (h_0)
- Column (4) Height of the Deck in the middle (h_1)
- Column (5) Ratio $L^2/8h_1$ in meters.
- Column (6) Ratio L/h_1
- Column (7) Distance between the Webs in meters.
- Column (8) Thickness of the Webs in meters.

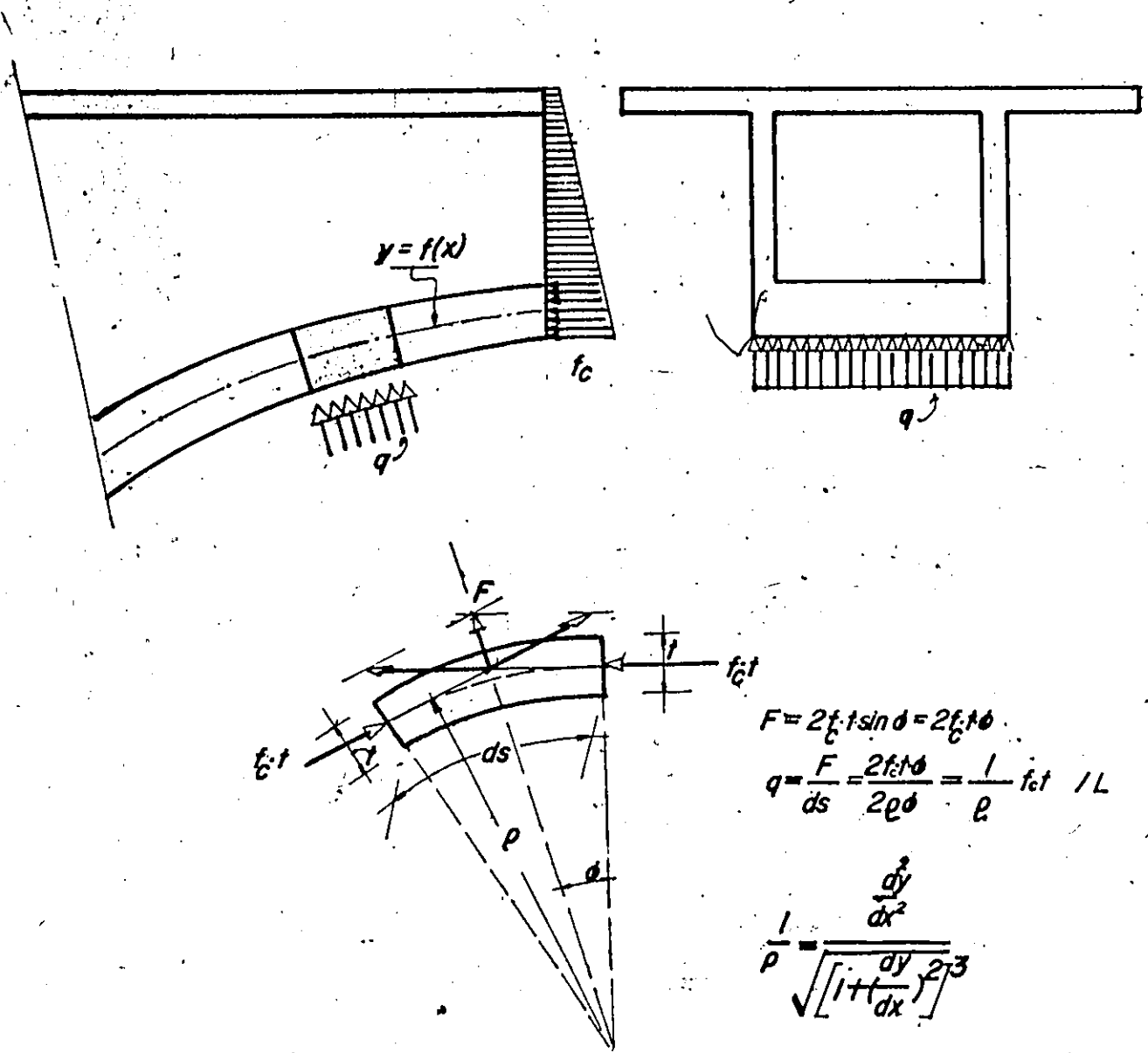
6.1.4 Transverse Deck Section

6.1.4.1 Bottom Slab

As already discussed, as the bottom slab is stressed by compression forces, except around the center of the span where positive moments occur, the thickness must be sufficient in order to keep the stresses within the permissible limits.

It can be either constant for bridges with small spans and parallel girders, or variable especially for haunched girders. In the latter case, the variation of thickness follows a parabolic law, as does the depth of the section and the dead load bending moments.

In addition to these longitudinal compression stresses the bottom slab has to resist transverse bending moments due to its own weight and the influence of the loadings on the top slab. Moreover, for haunched girders an additional bending moment is created by an upward loading due to the camber and the compression forces of the bottom slab (Fig. 6.8, Ref. 6.7).



$$F = 2f_c t \sin \theta = 2f_c t \theta$$

$$q = \frac{F}{ds} = \frac{2f_c t \theta}{2\rho\theta} = \frac{1}{\rho} f_c t \quad /L$$

$$\frac{1}{\rho} = \frac{\frac{dy}{dx}}{\sqrt{\left[1 + \left(\frac{dy}{dx}\right)^2\right]^{3/2}}}$$

Fig G.8 Upward forces in the bottom slab

6.1.4.2 Top Slab

The top slab is stressed longitudinally by small compression forces, except around the center where for continuous bridges positive moments occur and the compression forces are higher.

Transversely, the top slab is a beam over two or more supports, the supports being the webs of the box, with one corbel on each side. It is clear that for economical reasons a study has to be made to find the best web spacing and thickness of the various parts of the slab. More often, the two cornbels have a variable thickness with haunches at the supports while the slab between the webs has either a constant thickness or a variable one with a parabolical intrados.

Since the longitudinal prestressing steel is placed in the top slab, this has to be considered in the choice of the thickness. In Grand'Mère Bridge, due to the large number of bars, the slab had a thickness of 15 inches (38.1 cm) near the piers, which was reduced to 12 inches (30.5 cm) and finally to 11 inches (27.9 cm) at the center where the number of bars was small.

6.1.4.3 Webs

The webs must be dimensioned for the shear stresses. Since these stresses are higher near the piers the thickness there can be increased. This is not necessary for haunched girders because the unit shear stresses are almost constant along the length of the deck.

In addition to the shear stress the webs are stressed by transverse bending moments created by the top slab loadings, the temperature differential or the transverse post-tensioning. Sometimes the thickness of the web can be dictated by the fact that the longitudinal prestressing steel is anchored in their lower part and so thicker webs are necessary to facilitate concrete pouring.

6.1.4.4 Haunches

The haunches, in addition to reducing the spans of the top slab for the transverse design (Fig. 6.9a), increasing the thickness of the upper part of the webs where higher transverse moments exist, and making rigid connections between the webs and the slabs, serve also for the placing of the stressing steel and facilitating the pouring. In

addition, the exterior haunches must comply with architectural demands (Fig. 6.9b).

6.1.4.5 Shear Keys

As was mentioned in paragraph 5.3.4 the prefabricated segments have a series of shear keys on the webs and the top slab, the first serving to resist the shear forces between the erection time and the hardening of the epoxy used in the joints and the second series serving to facilitate the positioning of one segment relative to the other. While the top slab keys do not require a special design, those of the webs have to be dimensioned to take the shear and to allow the forces acting on the joint to compress the whole section (Ref. 6.1).

6.1.4.6 Segment Length

The choice of the segment length depends on the erection system and the available equipment. As all the design checks are made at each joint, this length must be known to the engineer before starting the final design.

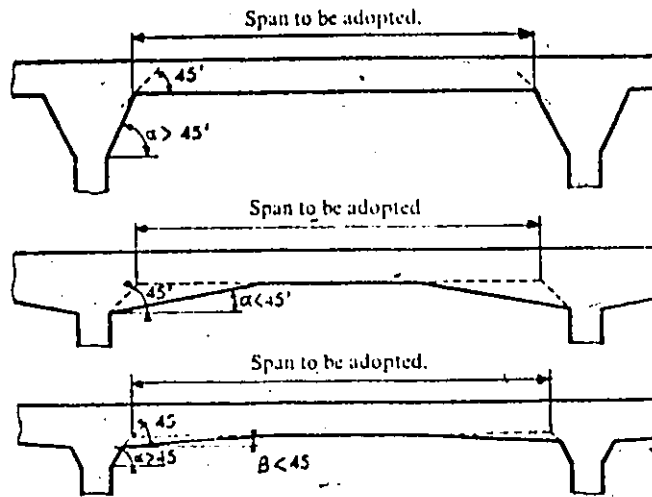


Fig. 6.9a. Span length of the top slab according to the French Code.

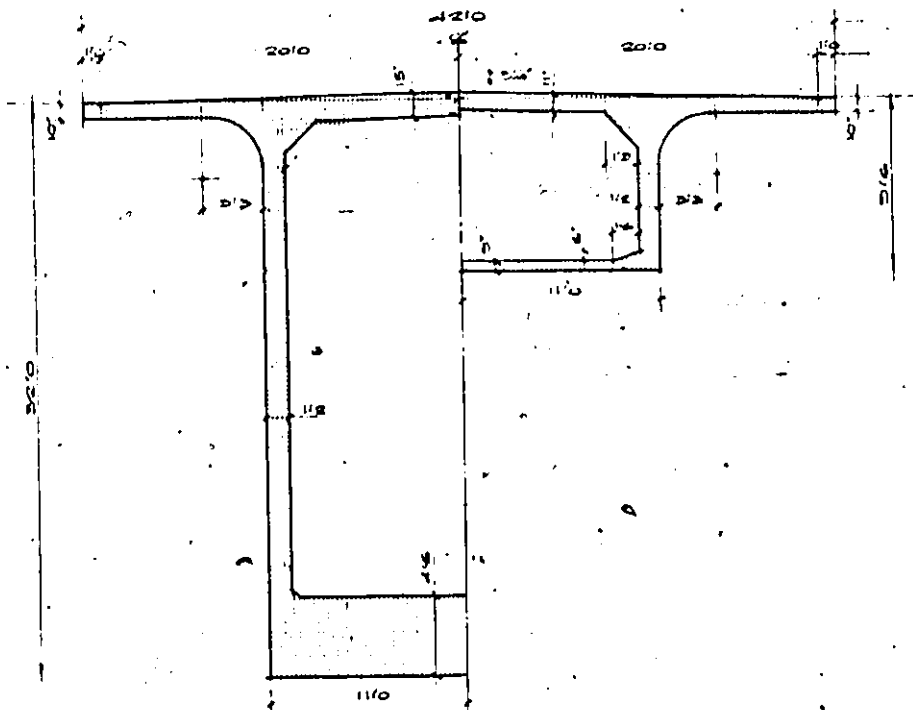


Fig.6.9b. Grand'Mère Bridge cross section

6.2 Design

The difference between the construction methods for the cantilever erection of a deck and the cast-in-place erection on a scaffolding requires a completely different design approach which takes into account all the intermediate steps of construction from the start of excavation to the finished bridge.

This design is longer and more difficult than that required for a simple cast-in-place deck and these difficulties are increased if the final statical system is different from that occurring during erection such as for continuous bridges.

For decks cast on scaffolding all the construction loads, fresh concrete, machinery, workmen, etc., are supported by the temporary structure and transmitted directly to the ground. The structure starts to support these loads only after the removal of the formwork and this occurs without any change of its statical system.

On the contrary, for the cantilever erected decks there are two distinct phases:

Phase one: This occurs during the construction and is subdivided into many secondary phases due to the modifications to the stresses throughout the erected structure.

Phase two: This takes place after the construction of the deck, that is once the deck has been made continuous.

The splitting of the first phase into many secondary phases can be better understood if we examine the construction cycle of one segment. Let us take for example the segment 201 of Grand'Mère Bridge (Fig. 6.10).

- (a) Phase 201a. The form-traveller is at its final position for the concreting of the segment 104, all the necessary adjustments for the camber are done and the mild and prestressing steel placed.
- (b) Phase 201b. Concreting of the segment. The load of the fresh concrete is supported by the form-traveller and transmitted to the finished part of the deck.
- (c) Phase 201c. A few hours later some prestressing steel is stressed to half its capacity because the

PREVIOUSLY COPYRIGHTED MATERIAL,
GRAPH ON LEAF 142,
NOT MICROFILMED

COPYRIGHT: GOUVERNMENT DU QUEBEC
MINISTERE DES TRANSPORTS
DIRECTION GENERALE DU GENIE

GRAPH OF GRAND MERE BRIDGE

concrete strength is still low. This stressing allows the transfer of the weight of the cast segment directly to the old deck and so frees the form-traveller.

- (d) Phase 201d. Moving forward of the form-traveller in order to cast the next segment.
- (e) Phase 201e. Stressing of all the prestressing steel of segment 201 to its full capacity.

Thus, just for the casting of one segment, the stresses throughout the completed portion of the structure were modified four times (phases b, c, d, e).

On Fig. 6.11 and 6.12 is a copy of the computer output with all the section forces and deformations for the phases 201 b and 201d.

For these reasons, the design phases of a cantilever bridge can be divided in three main groups:

- (a) Design and checking for the construction stages and the closure operation.

GRANDMÈRE BRIDGE - CONSTRUCTION STAGES MAY 4, 1976
MEMBER FORCES AND NODAL DEFORMATIONS LOADING 1 FORCES AT END OF CONST STA

POINT	H. DISPL.	V. DISPL.	ROTATION	MOMENT	SHEAR
1R	0.000107046	0.017504316	-0.000696117	0.0	0.0
2L	0.000107046	0.010045476	-0.000020870	-4752.28	-690.54
2R	0.000107046	0.010045476	-0.000020870	-4752.28	1110.82
3L	0.000107046	0.013946642	0.000105746	3794.80	-121.61
3R	0.000107046	0.013946642	0.000105746	3794.80	-121.61
4L	0.000107046	0.013173044	-0.000201630	207.78	-653.20
4R	0.000107046	0.013173044	-0.000201630	207.78	262.76
5L	0.000107046	0.009427682	-0.000113333	-1099.90	-394.76
5R	0.000107046	0.009427682	-0.000113333	-1099.90	333.62
6L	0.000107046	0.009029258	0.000033410	-1064.12	-331.28
6R	0.000107046	0.009029258	0.000033410	-1064.12	275.31
7L	0.000107046	0.010226436	0.000099541	-2268.40	-396.97
7R	0.000107046	0.010226436	0.000099541	-2268.40	290.04
8L	0.000107046	0.012791868	0.000132880	-3251.83	-389.62
8R	0.000107046	0.012791868	0.000132880	-3251.83	370.60
9L	0.000107046	0.014126174	-0.000081109	-4712.16	-489.36
9R	0.000107046	0.014126174	-0.000081109	-4712.16	238.14
10L	-0.000000006	0.011314984	0.000016948	-521.99	-592.78
10R	-0.000000006	0.011314984	0.000016948	-521.99	737.15
11L	-0.000022349	0.011608765	0.000085083	3058.29	627.79
11R	-0.000022349	0.011608765	0.000085083	3058.29	627.79
12L	-0.000040372	0.011971489	0.000093199	7082.39	301.35
12R	-0.000040372	0.000449355	0.000012879	-1495.20	301.35
13L	-0.000040372	0.000501098	0.000013322	0.00	0.0

Fig. 6.II Computer output for Construction Stage IOIb (or 20Ib) of Grand'Mère Bridge.

1 of

TRANSVERSE NODAL DEFORMATIONS LOADING 1 FORCES AT END OF CONST STAGE 101B

H. DISPL.	V. DISPL.	ROTATION	MOMENT	SHEAR	AX. FOR
0.000107046	0.017504316	-0.000696117	0.0	0.0	0.
0.000107046	0.010045476	-0.000020870	-4752.28	-690.54	0.
0.000107046	0.010045476	-0.000020870	-4752.28	1110.82	0.
0.000107046	0.013946642	0.000105746	3794.80	-121.61	0.
0.000107046	0.013946642	0.000105746	3794.80	-121.61	0.
0.000107046	0.013173044	-0.000201630	207.78	-653.20	0.
0.000107046	0.013173044	-0.000201630	207.78	262.76	0.
0.000107046	0.009427682	-0.000113333	-1099.90	-394.76	0.
0.000107046	0.009427682	-0.000113333	-1099.90	333.62	0.
0.000107046	0.009029258	0.000033410	-1064.12	-331.28	0.
0.000107045	0.009029258	0.000033410	-1064.12	275.31	0.
0.000107046	0.010226436	0.000099541	-2268.40	-396.97	0.
0.000107046	0.010226436	0.000099541	-2268.40	290.04	0.
0.000107046	0.012791868	0.000132880	-3251.83	-389.62	0.
0.000107046	0.012791868	0.000132880	-3251.83	370.60	0.
0.000107046	0.014126174	-0.000061109	-4712.16	-489.36	0.
0.000107046	0.014126174	-0.000061109	-4712.16	238.14	0.
0.000000006	0.011314984	0.000016948	-521.99	-592.78	-514.
0.000000006	0.011314984	0.000016948	-521.99	787.15	-514.
0.000022349	0.011608765	0.000085083	3058.29	627.79	-514.
0.000022349	0.011608765	0.000085083	3058.29	627.79	-514.
0.000040372	0.011971489	0.000093199	7082.39	301.35	-514.
0.000040372	0.000449355	0.000012879	-1495.20	301.35	0.
0.000040372	0.000561098	0.000013322	0.00	0.0	0.

g. 6.II Computer output for Construction Stage IOIb (or 20Ib) of Grand'Mère Bridge.

2 of 2

MEMBER FORCES AND NODAL DEFORMATIONS - LOADING 1 - FORCES AT END OF CONST. STAG

POINT	H. DISPL.	V. DISPL.	ROTATION	MOMENT	SHEAR
1R	0.000187331	0.017547300	-0.000697175	0.0	0.0
2L	0.000187331	0.010071769	-0.000021929	-4752.28	-690.54
2R	0.000187331	0.010071769	-0.000021929	-4752.28	1115.53
3L	0.000187331	0.013956163	0.000104647	3869.47	-116.90
3R	0.000187331	0.013956163	0.000104647	3869.47	-116.90
4L	0.000187331	0.013173044	-0.000202772	321.73	-648.48
4R	0.000187331	0.013173044	-0.000202772	321.73	253.93
5L	0.000187331	0.009404045	-0.000114513	-1162.49	-403.59
5R	0.000187331	0.009404045	-0.000114513	-1162.49	322.97
6L	0.000187331	0.008983673	0.000032463	-1339.77	-341.93
6R	0.000187331	0.008983673	0.000032463	-1339.77	261.61
7L	0.000187331	0.010166913	0.000099134	-2818.09	-410.67
7R	0.000187331	0.010166913	0.000099134	-2818.09	272.34
8L	0.000187331	0.012732502	0.000133376	-4155.52	-407.32
8R	0.000187331	0.012732502	0.000133376	-4155.52	349.38
9L	0.000187331	0.014099084	-0.000078893	-6146.56	-510.58
9R	0.000187331	0.014099084	-0.000078893	-6146.56	215.52
10L	-0.000000010	0.011327036	0.000017993	4054.15	-615.40
10R	-0.000000010	0.011327036	0.000017993	4054.15	787.15
11L	-0.000039110	0.011622999	0.000084912	7666.55	627.79
11R	-0.000039110	0.011622999	0.000084912	7666.55	627.79
12L	-0.000070650	0.011983085	0.000092051	9515.59	501.35
12R	-0.000070650	0.000460955	0.000011731	938.00	501.35
13L	-0.000105539	0.000572667	0.000010267	6006.08	0.0

Fig. 6.12 Computer output for Construction Stage 10Id (or 20Id) of Grand'Mère Bridge.

1 of 1

ES AND NODAL DEFORMATIONS LOADING 1 FORCES AT END OF CONST STAGE 1010

H.DISPL.	V.DISPL.	ROTATION	MOMENT	SHEAR	AX.FOR
0.000187331	0.017547369	-0.000697175	0.0	0.0	0.
0.000187331	0.010071769	-0.000021929	-4752.28	-690.54	0.
0.000187331	0.010071769	-0.000021929	-4752.28	1115.53	0.
0.000187331	0.013956163	0.000104647	3869.47	-116.90	0.
0.000187331	0.013956163	0.000104647	3869.47	-116.90	0.
0.000187331	0.013173044	-0.000202772	321.73	-648.48	0.
0.000187331	0.013173044	-0.000202772	321.73	253.93	0.
0.000187331	0.009404045	-0.000114513	-1162.49	-403.59	0.
0.000187331	0.009404045	-0.000114513	-1162.49	322.97	0.
0.000187331	0.008983673	0.000032463	-1339.77	-341.93	0.
0.000187331	0.008983673	0.000032463	-1339.77	261.61	0.
0.000187331	0.010166913	0.000099134	-2818.09	-410.67	0.
0.000187331	0.010166913	0.000099134	-2818.09	272.34	0.
0.000187331	0.012732502	0.000133376	-4155.52	-407.32	0.
0.000187331	0.012732502	0.000133376	-4155.52	349.38	0.
0.000187331	0.014099084	-0.000078893	-6146.56	-510.58	0.
0.000187331	0.014099084	-0.000078893	-6146.56	215.52	0.
-0.000000010	0.011327036	0.000017993	4054.15	-615.40	-899.
-0.000000010	0.011327036	0.000017993	4054.15	787.15	-899.
-0.000039110	0.011622999	0.000084912	7666.55	627.79	-899.
-0.000039110	0.011622999	0.000084912	7666.55	627.79	-899.
-0.000070650	0.011983085	0.000092051	9515.59	501.35	-899.
-0.000070650	0.000480955	0.000011731	938.00	501.35	-385.
-0.000105539	0.000572667	0.000010267	6006.08	0.0	-385.

Fig. 6.I2 Computer output for Construction Stage 101d (or 201d) of Grand'Mère Bridge.

2 of 2

- (b) Design for the period after the construction.
- (c) Design of those parts of the deck which are not influenced by the construction method.

In the following, each group will be examined.

6.2.1 Design for the Construction Period

6.2.1.1 Substructure

The substructure has to be designed and checked for all the construction stages and the stability of the deck-substructure system insured.

6.2.1.2 Stresses in the Deck

All the stresses have to be checked at each joint for the various construction phases.

6.2.1.3 Ultimate Capacity of the Bridge

The ultimate capacity of each section must be checked at each joint and for the most unfavourable load combination.

At the Grand'Mère Bridge, the ultimate capacity was calculated with a compression prism different from that given by the AASHO code. The prism was formed by a straight line and a parabola (Fig. 6.13a) as prescribed by the F.I.P. code (Ref. 6.9). To facilitate the design, a special graph was drawn (Fig. 6.13b) which immediately gives for each concrete strain, the prism volume (magnitude of the compression force in the compressed zone) as well as the exact position of this compression force.

6.2.1.4 Computation and Checking of the Deflections of the Cantilevers

It is evident that the elastic line of the cantilevers changes each time there is a change of the loading (Fig. 6.14). So, during the construction of segment 116 the cantilever end at the phase 116a had a deflection of 0.776 foot upwards while at phase 116b the deflection was 0.615 foot, also upwards, almost 2 inches lower than in phase a.

In order that the final profile correspond to the one desired, all the intermediate elastic lines have to be calculated before the construction. This computation is

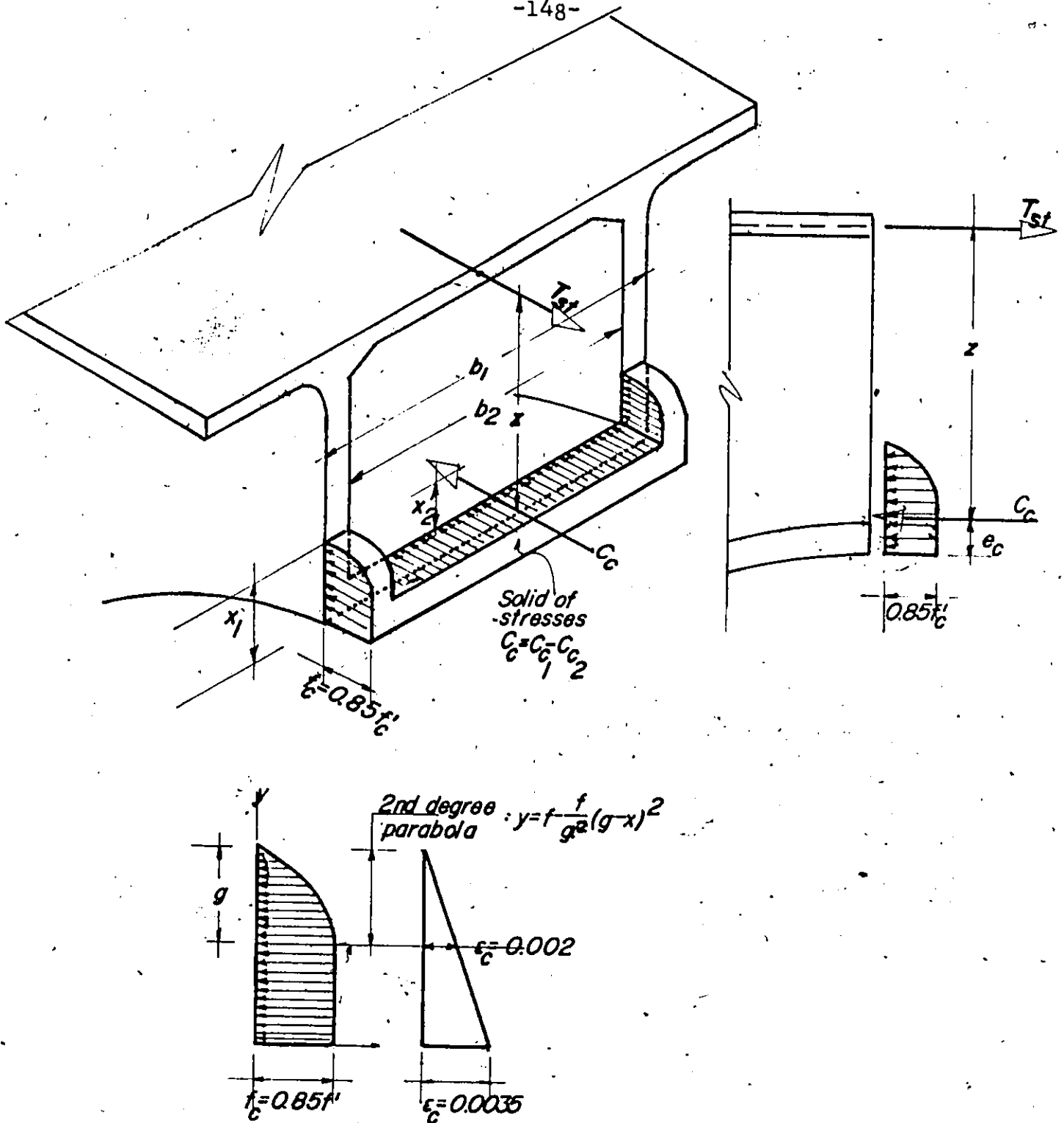
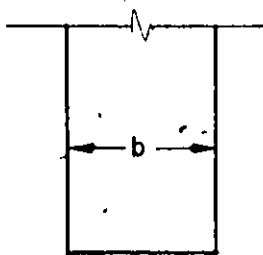
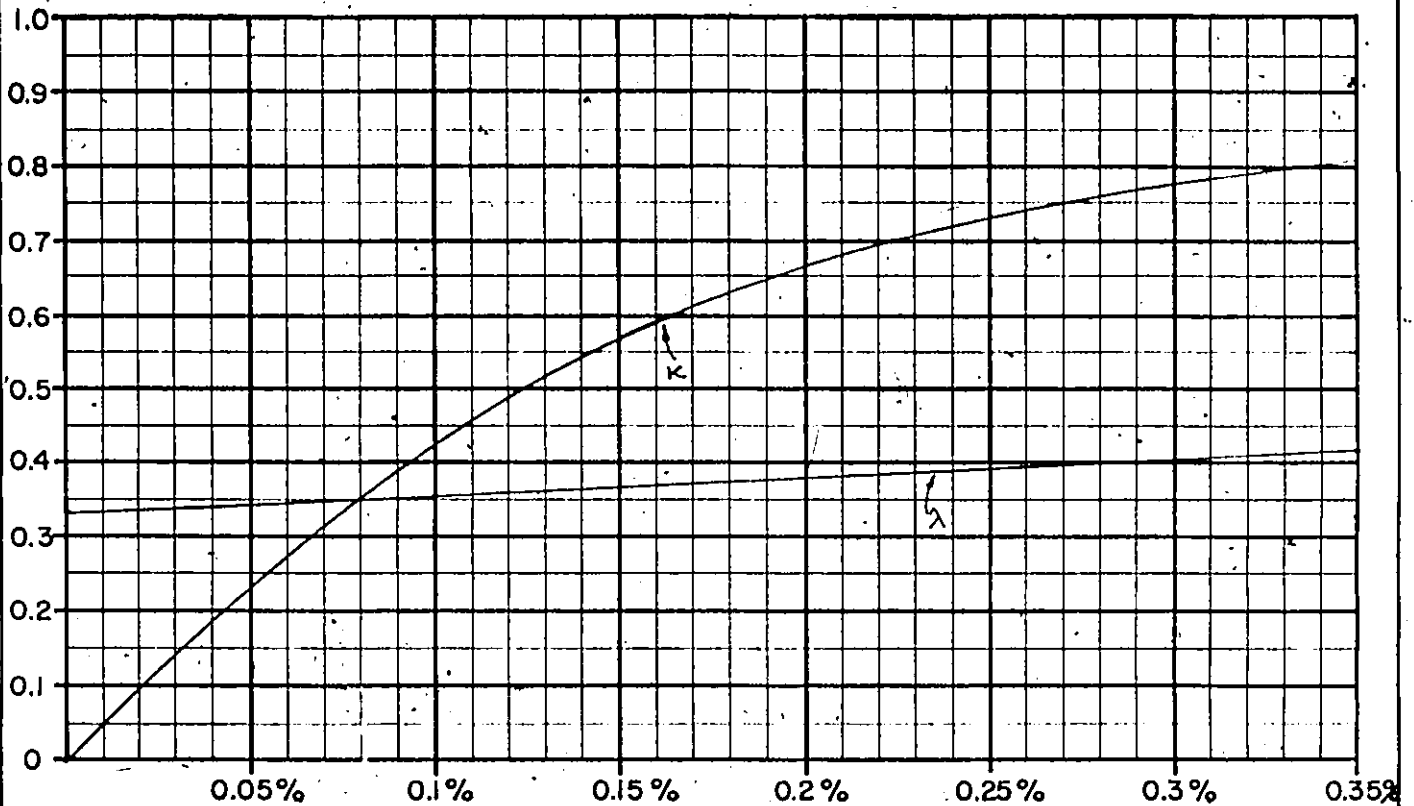
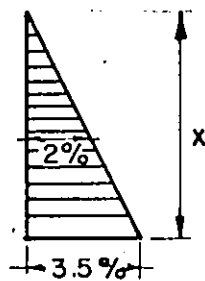


Fig. 6.13a Stress-strain diagram for the U.S.D. according to the F.I.P

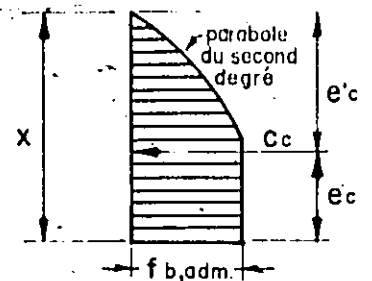
PRISME DE COMPRESSION DU BETON A L'ETAT LIMITE



COUPE



DEFORMATION ϵ_c



VOLUME DE COMPRESSION

VOLUME DU PRISME DE COMPRESSION $C_c = K \times b \times X \times f_{b, adm.}$

CENTRE DE GRAVITE

$e_c = \lambda \times X$

$e'_c = \lambda' \times X$

Fig. 6.I3b Graph for the U.S.D.
according to the F.I.P. Code.

PREVIOUSLY COPYRIGHTED MATERIAL,
LEAF 150,
NOT MICROFILMED

COPYRIGHT: GOUVERNEMENT DU QUEBEC
MINISTERE DES TRANSPORTS
DIRECTION GENERALE DU GENIE

GRAPH OF THE GRAND'MERE BRIDGE

done from the final profile of the completed bridge and by working backwards through all the construction stages.

Due to the different age of each segment, the choice of the modulus of elasticity for the deflection computation is more difficult for the cast-in-place cantilever method than for the prefabricated segments method.

This design can be performed either by taking into account the real modulus of elasticity of each segment according to its age or to simplify the task, by assuming a mean value for it, as it was done in Grand'Mère Bridge. (The value used was 550,000 ksf compared to 620,000 ksf of the final stage.)

The effect of creep upon the elastic curve of the bridge has to be taken into account for this plastic deformation can be as much as two times the elastic one. The creep factor ϕ has to be evaluated using the formulas of the code. In Grand'Mère, the formula $\phi = \phi_0 C_1 C_2 C_3 C_4$ of the German code was used and the value ϕ was found equal to 2.

In this formula the variables are:

ϕ_0 : the basic creep factor

C_1 : factor reflecting the influence of the age of

- concrete when the load is applied
- C₂: factor reflecting the influence of the composition of concrete (water/cement)
- C₃: factor reflecting the influence of the dimensions of the structural member
- C₄: factor reflecting the influence of time depending advance of the creep.

As erection progresses, the deflections are verified and compared with the theoretical ones. If some modifications have to be done this is possible either by adjusting the form-traveller before the next pour for the cast-in-place method, or by inserting, during erection, stainless steel wire shims of a maximum thickness of 1/16 inch (1.6 mm) in the epoxy joints for the prefabricated method.

6.2.1.5 Evaluation of Stresses Due to the Closure Operation (Ref. 6.12, 6.13, 6.7)

This includes all the final procedures performed in order to complete the bridge and their effect on the section forces of the deck. These procedures include the

removal of temporary supports or temporary prestressing forces used to stabilize the hammerhead, the removal of the auxiliary piers and stays used to reduce the stresses in the cantilevers or the removal of the form-traveller.

Another operation performed in this last stage is the closing operation used sometimes to artificially induce section forces in the deck.

In fact, the bending moments created during the construction will undergo a redistribution with time all over the span due to the creep effect. This has as a result of decreasing the negative moment over the pier and the causing of a positive moment in the center of the span. But, it is more economical to reduce these moments before the structure is made continuous in order to save the additional steel which must be placed over the piers to take the negative moments due to the dead loads placed after the closure of the bridge (Fig. 6.15) (asphalt, parapet, etc.).

The reduction of the negative moments can be achieved by introducing positive moments in the deck. These moments can be created by lifting, for example, some bearings (Fig. 6.16a) or by tensioning some prestressing steel in the open system as in Grand'Mère Bridge.

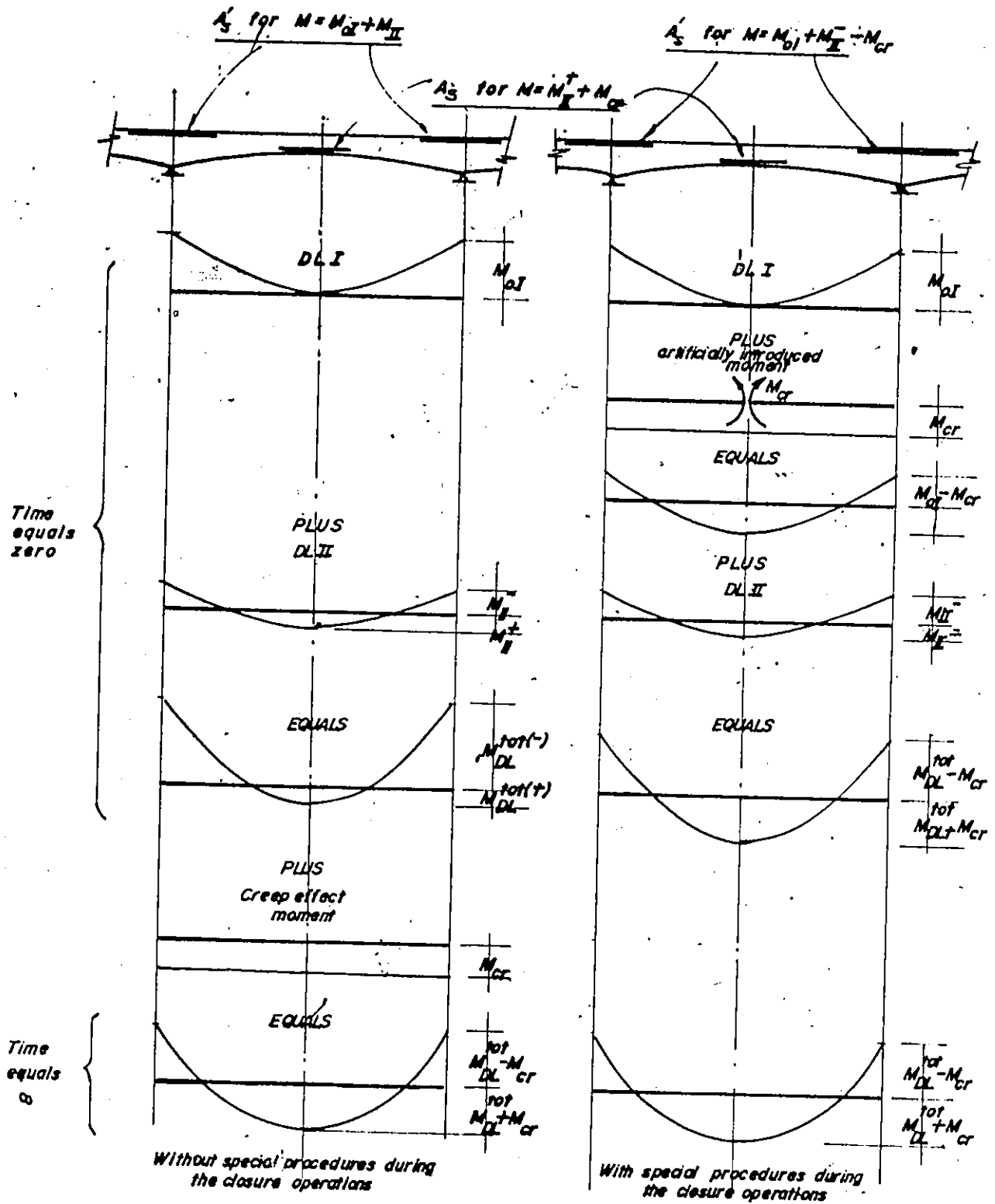


Fig 6.15 Closure operations

For this, in the 3 foot gap left between the two cantilevers the top slab only was cast hinged to the top slab of the finished deck. Once the concrete strength was obtained, some bars of the bottom slab were tensioned so creating a positive moment equal to P.e. (Fig. 6.16b).

6.2.2 Design for the After the Construction Period

6.2.2.1 Computation of the Creep Effect on the Section Forces

As already mentioned, the creep will cause a redistribution of the negative moments created during the construction in the final continuous structure.

This is a particularity of structures cast according to a statical system (cantilever) that is completely different from the final one (continuous girder). This was already explained in Chapter 5.

6.2.2.2 Computation of the Secondary Moments Induced by the Post-Tensioning

These moments are created both by the steel stressed during the construction due to the creep effect

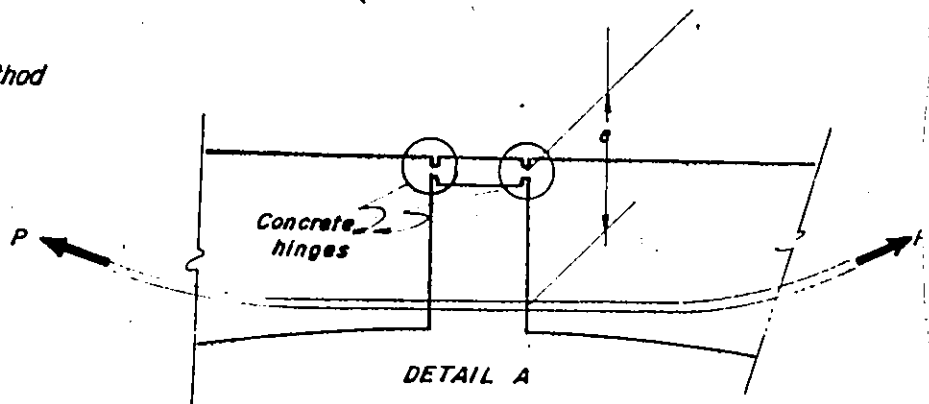
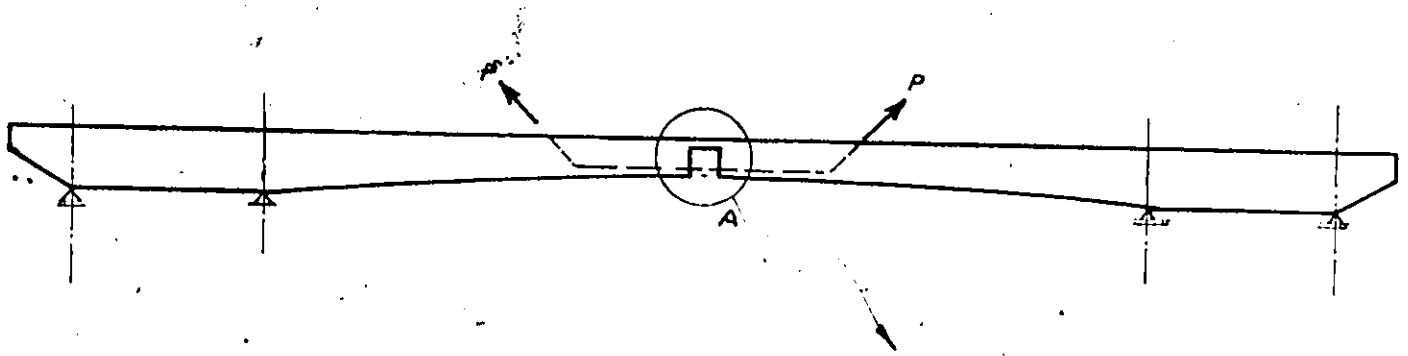
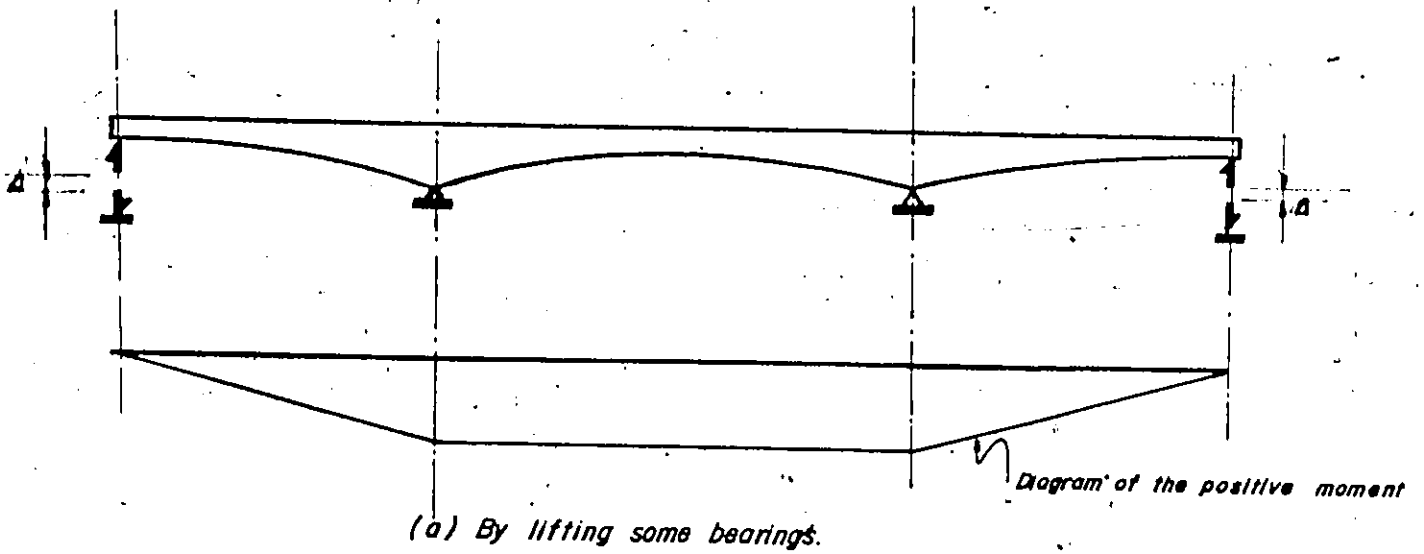


Fig. G.16 Methods to introduce positive moments in the structure.

and also by that stressed after the deck is continuous.

6.2.3 Design not Influenced by the Construction Method

6.2.3.1 Design of the Deck in the Transverse Direction

The deck is designed transversely as a closed frame using any of the known methods of strength of materials. The various loads are the dead load of the section, the additional dead load due to asphalt, parapet, etc., the live load on the top slab, the temperature differential between the inside and outside of the section, the effect of the post-tensioning of the top slab and sometimes the wind.

For sections having more than one box, the design is complicated because the rigidity of the common slab and the influence of one box on the other has to be accounted for.

The design for the live load must be done by the elastic methods and not with the empirical formulas given by the code. These methods make use of charts of influence surfaces (analogous to the influence line but for two dimensions).

The design procedure consists of placing the wheel "footprint" on the most unfavourable position on the chart and evaluating the volume of the prism between the ordinates of the influence surface and the wheel "footprint" (Fig.6.17). This method permits the designer to take into account the slab rigidity and the support conditions. Thus, for the slab between the webs, the edges can be assumed fixed and the fixed end moments are calculated with the influence surfaces. Then the designer can do the moment distribution in the frame according to the Cross method or any other one.

The writer found two books treating the influence surface method adequately. The first one (Ref. 6.10) deals with slabs of constant depth while the other (Ref. 6.11) deals both with constant and variable depth, the variation done according to specific laws.

Most of the time the top slab is transversally post-tensioned and in this case, bending moments are induced in the webs of the box due to the elastic shortening of the top slab. The creep and shrinkage of the concrete will reduce these moments with time by almost 30% (Ref. 6.6).

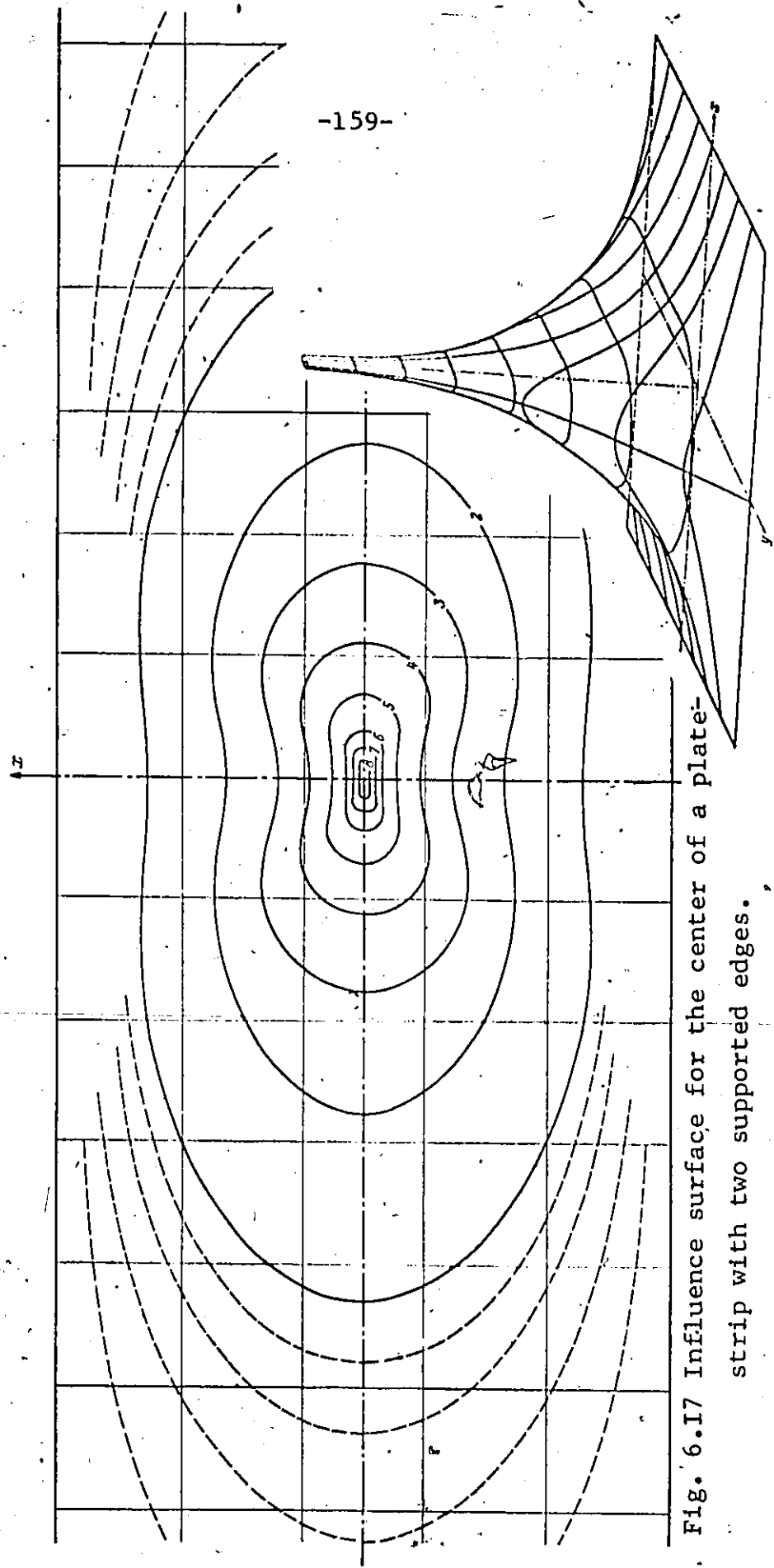


Fig. 6.I7 Influence surface for the center of a plate-strip with two supported edges.

6.2.3.2 Diaphragms

Due to the great transverse rigidity of the box, diaphragms are not necessary except over the piers where they serve to keep the section undeformed and help to transmit the reaction from the webs to the bearings.

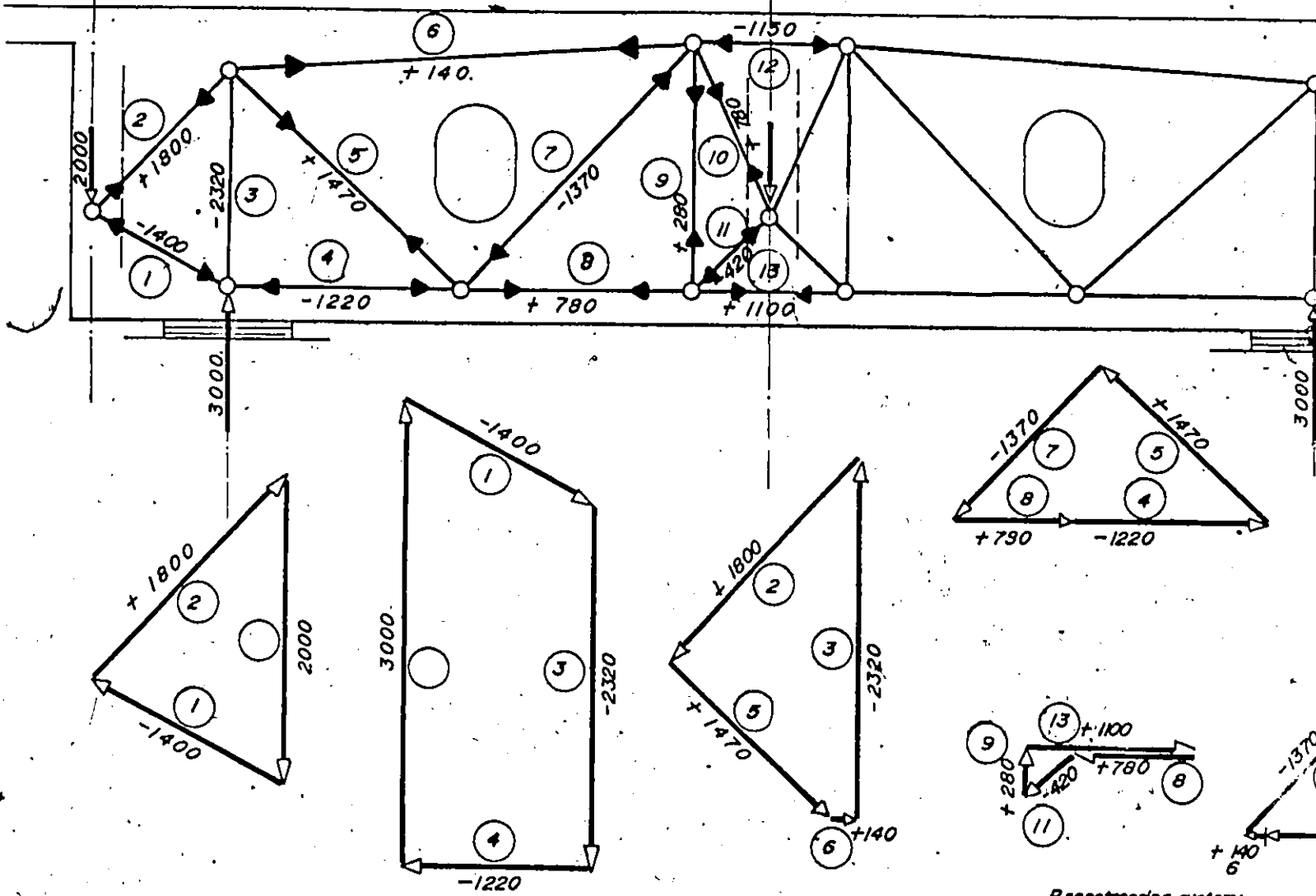
The truss theory is one method used to design these diaphragms.

A fictitious truss is chosen in the plane of the diaphragm of the member through which the loads will be transferred. The tensile members are represented by the steel to be placed while the compression members are represented by the concrete. In Fig. 6.18, an example of this method is given.

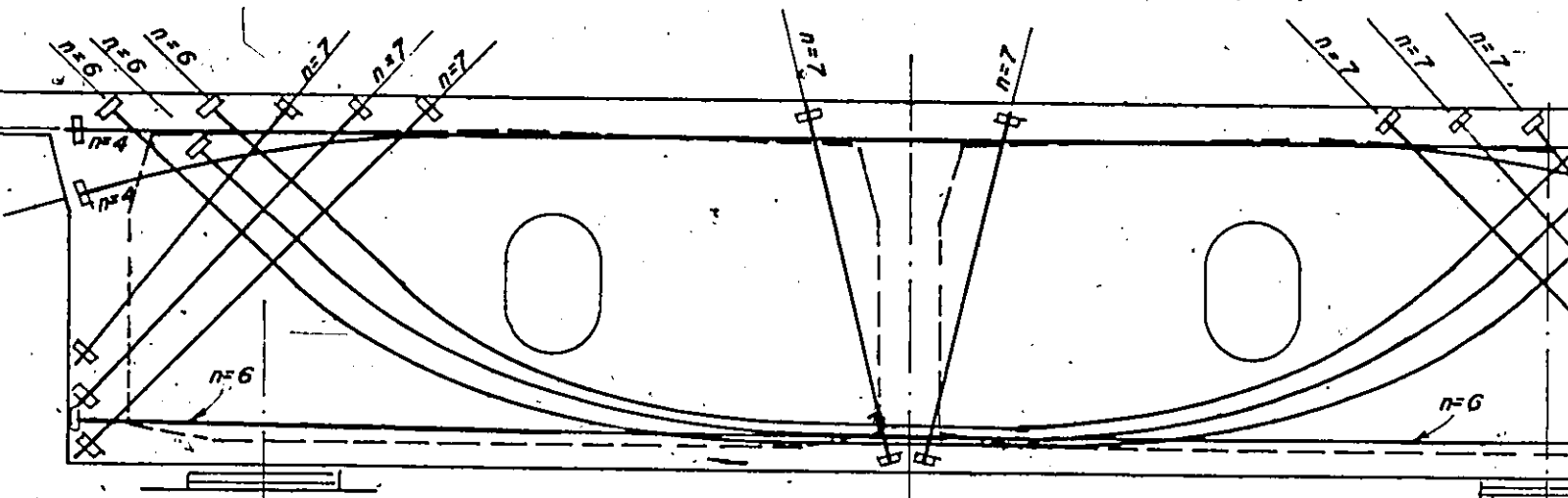
In addition to the forces coming through the webs, one must also take into account the tensile stresses resulting from the longitudinal compression forces of the bottom slab in the case of haunched decks (Fig. 6.19).

6.2.3.3 Shear Forces and Torsion

The shear design has to be done for the working conditions as well as for the ultimate loads.



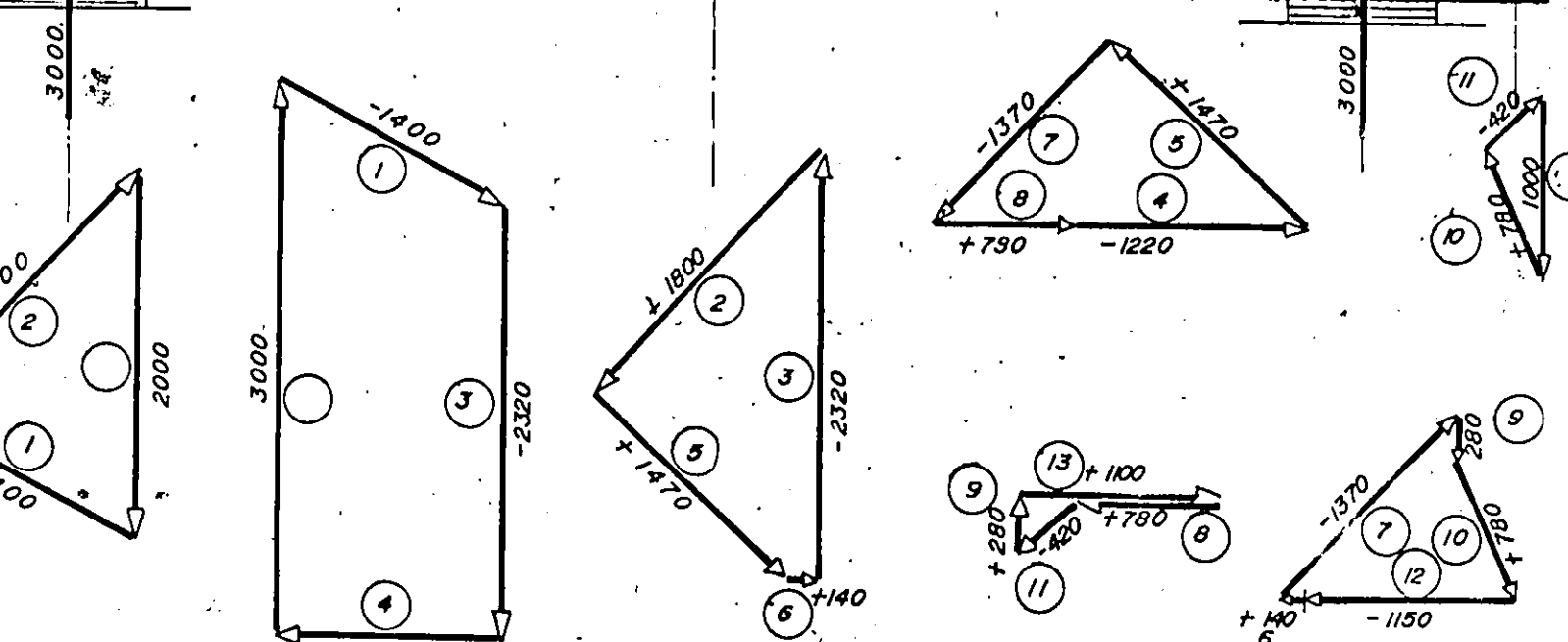
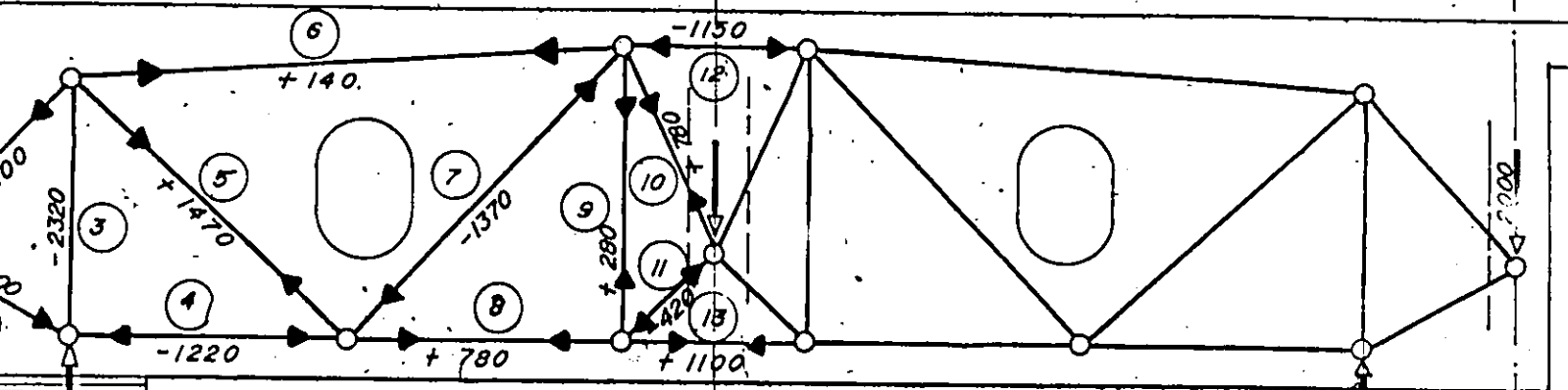
Pressurizing system:
DYWIDAG bars $\phi 32\text{mm}$ $f_y = 150$



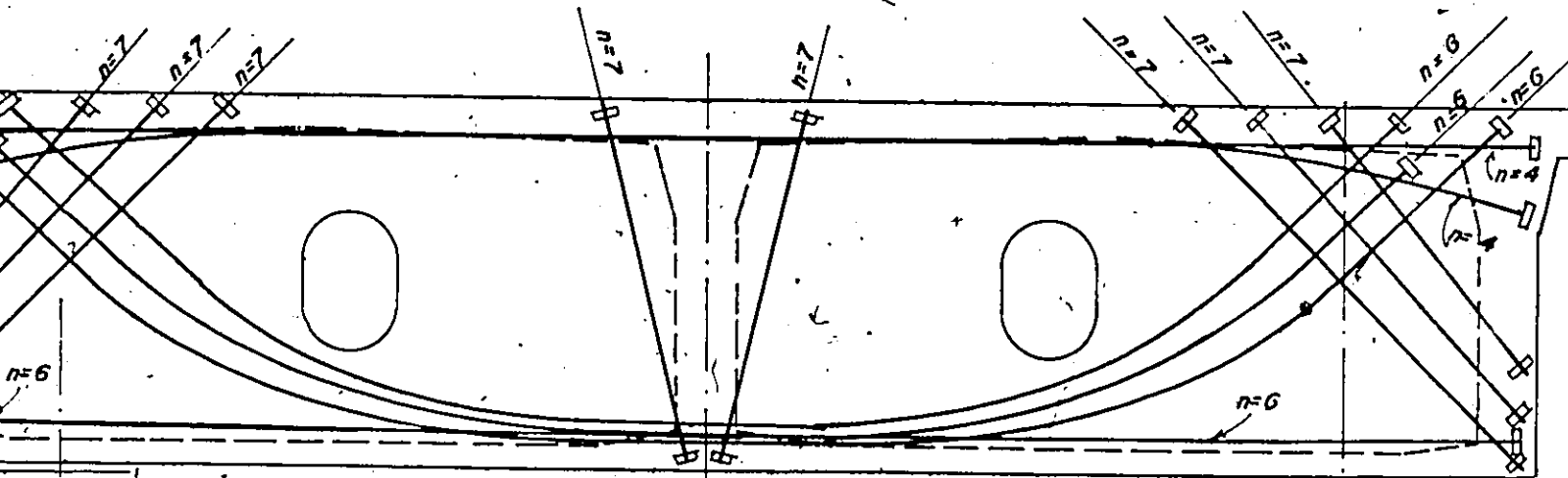
To find the necessary number of prestressing bars
divide the forces found by the ultimate force of a bar
and multiply the quotient by the U.S.D. load factor.
Example: Member 2 $n = 1800 \text{ kips} \times 1.8 / 150 = 21.6$
Member 5 $n = 1470 \text{ kips} \times 1.8 / 150 = 17.6$

1 of

Fig G.18 Diaphragm design using the truss analogy method



Pressing system:
DYWIDAG bars $\phi 32\text{mm}$ $f = 150\text{ ksi}$



To find the necessary number of prestressing steel, one must divide the forces found by the ultimate force of the prest. unit and multiply the quotient by the U.S.D. load factors.
 Example: Member 2 $n = 1800\text{ kips} \times 1.8 : 150 = 21.6$ (3X7)
 Member 5 $n = 1470 \text{ " } \times 1.9 : 150 = 17.6$ (3X6)

Fig G.18 Diaphragm design using the truss analogy method

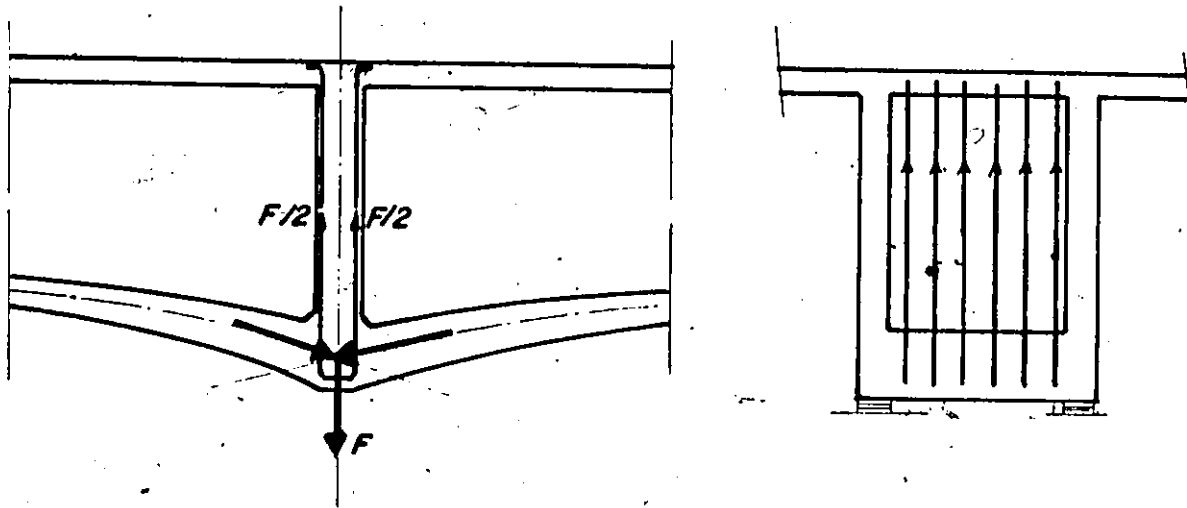


Fig G.9 Tensile forces in the diaphragms due to the bottom slab

The stresses created by the shear force must be added to those created by the torsion for the most unfavourable condition; that is, "maximum shear force with the resulting torsion" or "maximum torsion with the resulting shear force".

The shear stresses due to torsion for a simple box with small crosswise dimensions compared to the span can be found by the classical formula of Bredt. The longitudinal distribution of torsional moment for a haunched girder is somewhat longer to find (Ref. 6.20).

6.3.3.4 Effect of Haunched Girders and of Prestressing Steel on the Shear Force

For a variable inertia deck, the shear force can either be reduced as in the case where the center line of the bottom deck forms a positive angle with the horizontal (Fig. 6.20) or increased if this angle is negative (Fig. 6.21). The shear force can also be reduced by the vertical component of the prestressing force when the longitudinal steel is anchored down in the webs (Fig. 6.22).

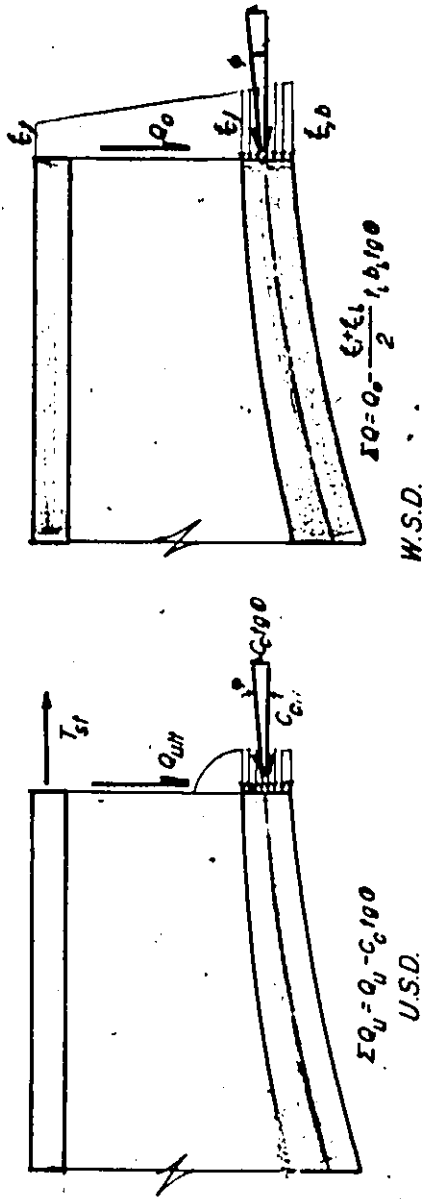


Fig. 6.20 Positive effect of haunched girders on shear force.

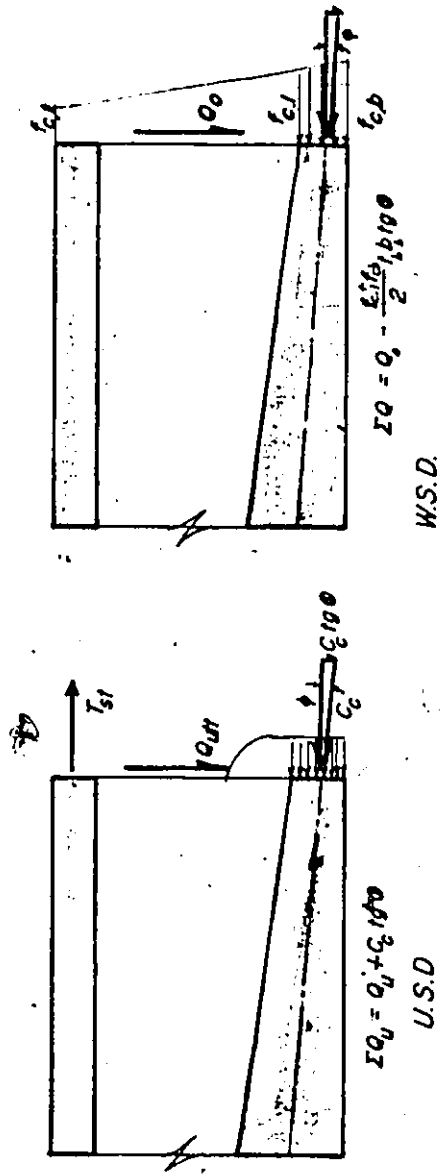


Fig. 6.21 Negative effect of haunched girders on shear force.

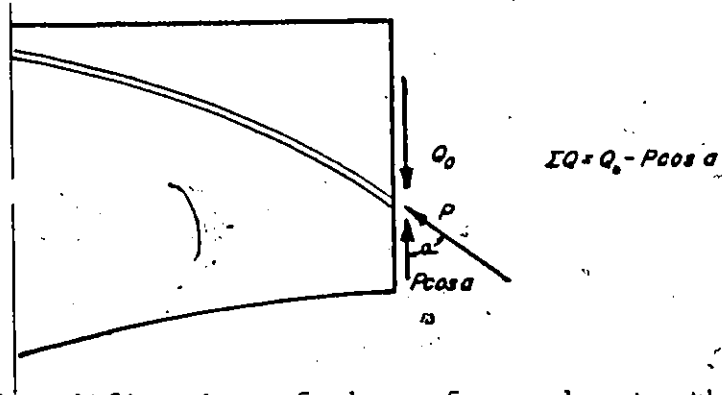


Fig. 6.22 Modification of shear force due to the prestressing steel.

CHAPTER 7

EXAMPLES OF BRIDGES ERECTED
BY THE CANTILEVER METHOD

In the following discussion, a brief description of bridges erected by the cantilever method will be given.

The choice of the examples is very arbitrary because there are so many bridges of this type that there are too many to choose from.

An attempt was made to have an example of each statical system already described, as well as bridges having a special feature during the erection.

Further details on these structures can be found in the articles given in the references from which these notes were taken.

7.1 Bendorf Bridge Over the Rhine (Federal Republic of Germany) (Fig. 7.1, Ref. 7.1)

With a main span of 208 m. (682.4 feet) this bridge was, at that time, the longest to be built in prestressed concrete using the cantilevering method and even today it is among the longest in this category.

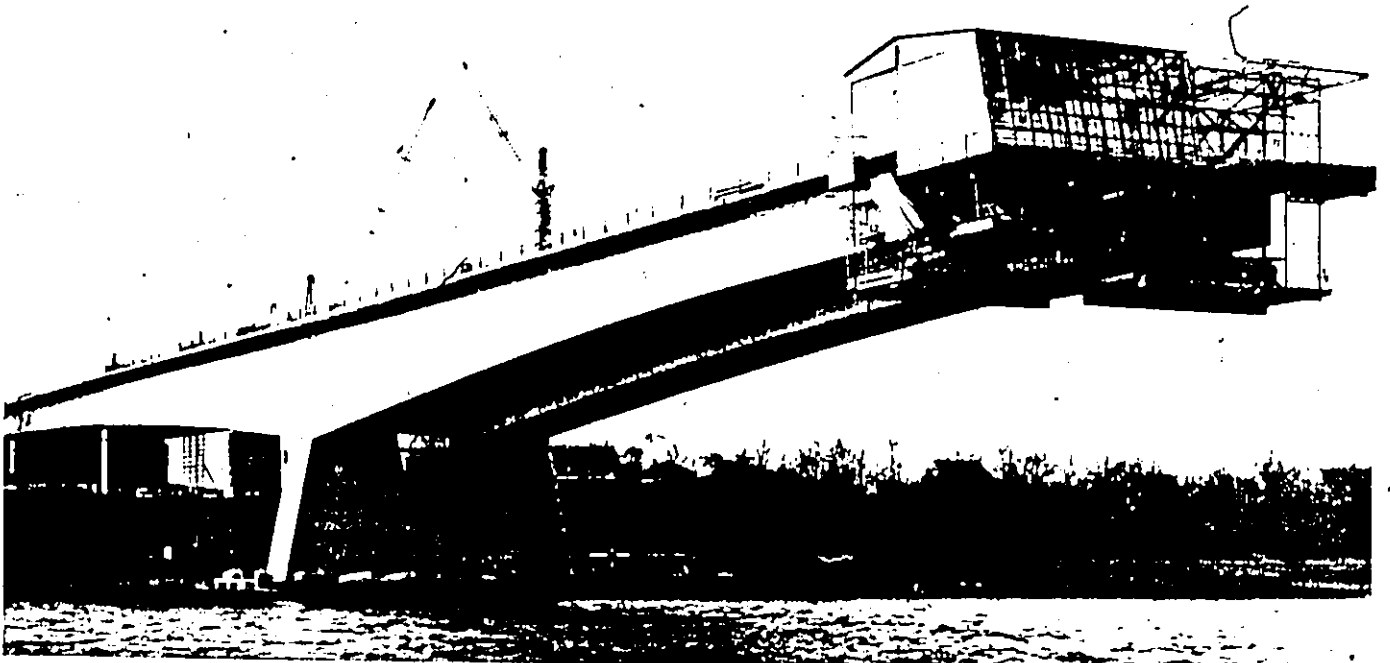
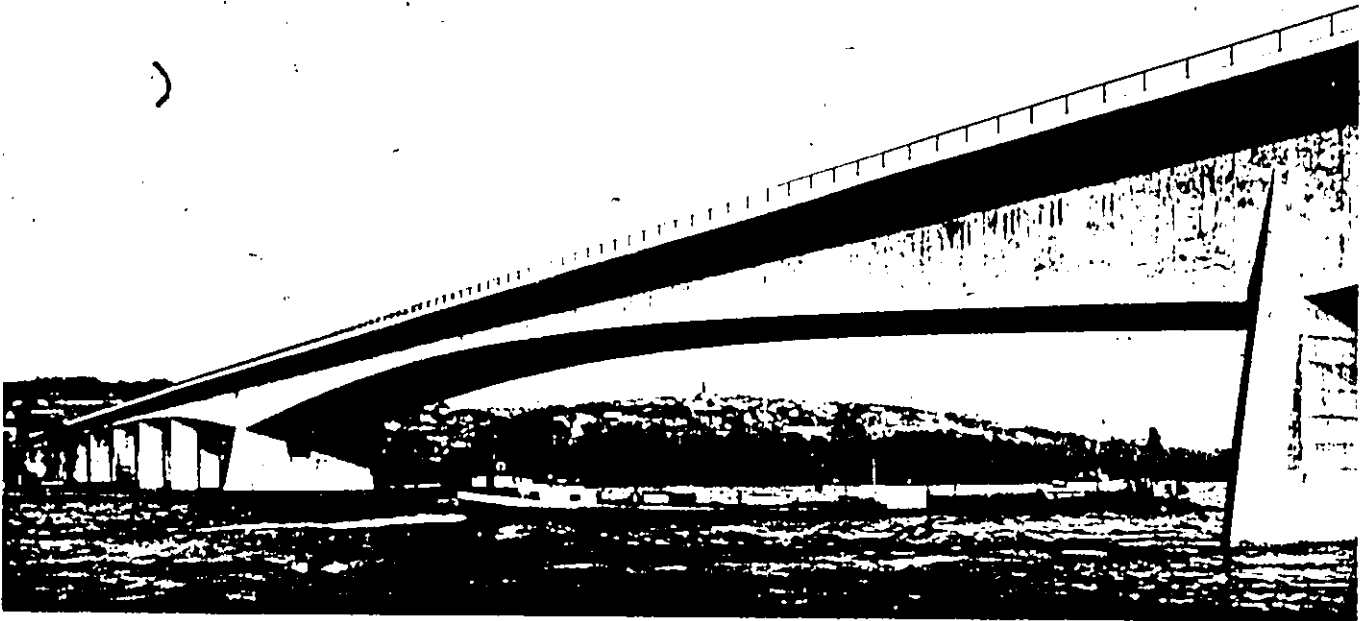


Fig. 7.I Bendorf Bridge over the Rhine,, W.Germany.
General view (top) and Under Construction (bottom).

The total length of 1,029.5 m (3,378 feet) from one abutment to the other was divided in two parts for the bids. The West part of 524.5 m (1,721 feet) long was designed and built by the companies Dyckerhoff & Widmann AG of Munich and Grun & Dilfinger of Mannheim, and the East part of 505 m (1,657 feet) long was designed and built by the company Wayss & Freitag of Frankfurt.

The first part is composed of a central span of 208 m (682.4 feet), with a hinge in the middle, framed by three spans on each side. Several methods were used for the construction; the conventional cantilevering method, the cantilevering with cable-stay and the cast-in-place on scaffolding. The deck is prestressed longitudinally, transversely and vertically (with prestressed stirrups inclined at 45°) by the Dywidag system of prestressed bars.

The second part consists of a viaduct of five spans between 41 m (134.5 feet) and 60 m (196.8 feet) built on scaffolding and a bridge of four spans between 47.85 m (157 feet) and 58 m (190.3 feet) built by cantilevering with cable-stay. The structure is prestressed with various prestressing systems: Vorspann-Technik, Freyssinet-Wayss & Freitag and finally Dywidag for the cantilevered part.

The cross section of the structure is comprised of a single box for each traffic direction. The total width is 30.86 m (101.2 feet) for four traffic lanes of 6.60 m (21.6 feet) each separated by a security rail of 3 m (9.8 feet) and a lane for cyclists and pedestrians plus a service foot-bridge.

The tender call was made at the beginning of 1960 and the structure was inaugurated in the summer of 1966.

7.2 Medway Bridge (Great Britain) (Ref. 7.2)

This bridge, almost 1,000 m (3,281 feet) long (Fig. 7.2), completed in the early sixties, consists of a three-span bridge (a main span of 152.5 m (500.3 feet) and two lateral spans of 95.25 m (302.7 feet) each) flanked by the East viaduct of 243.08 m (797.5 feet) long (30.48, 4 x 33.53, 39.28, 39.20 m) and the West viaduct of 411.48 m (1,350 feet) (30.48, 4 x 33.53 and 6 x 41.15 m). The main span is of the cantilever type with a suspended span of 30.4 m (99.7 feet) long. The bridge was the result of a competition between various solutions, the final design was adopted because of the appealing aesthetics of the bridge. The

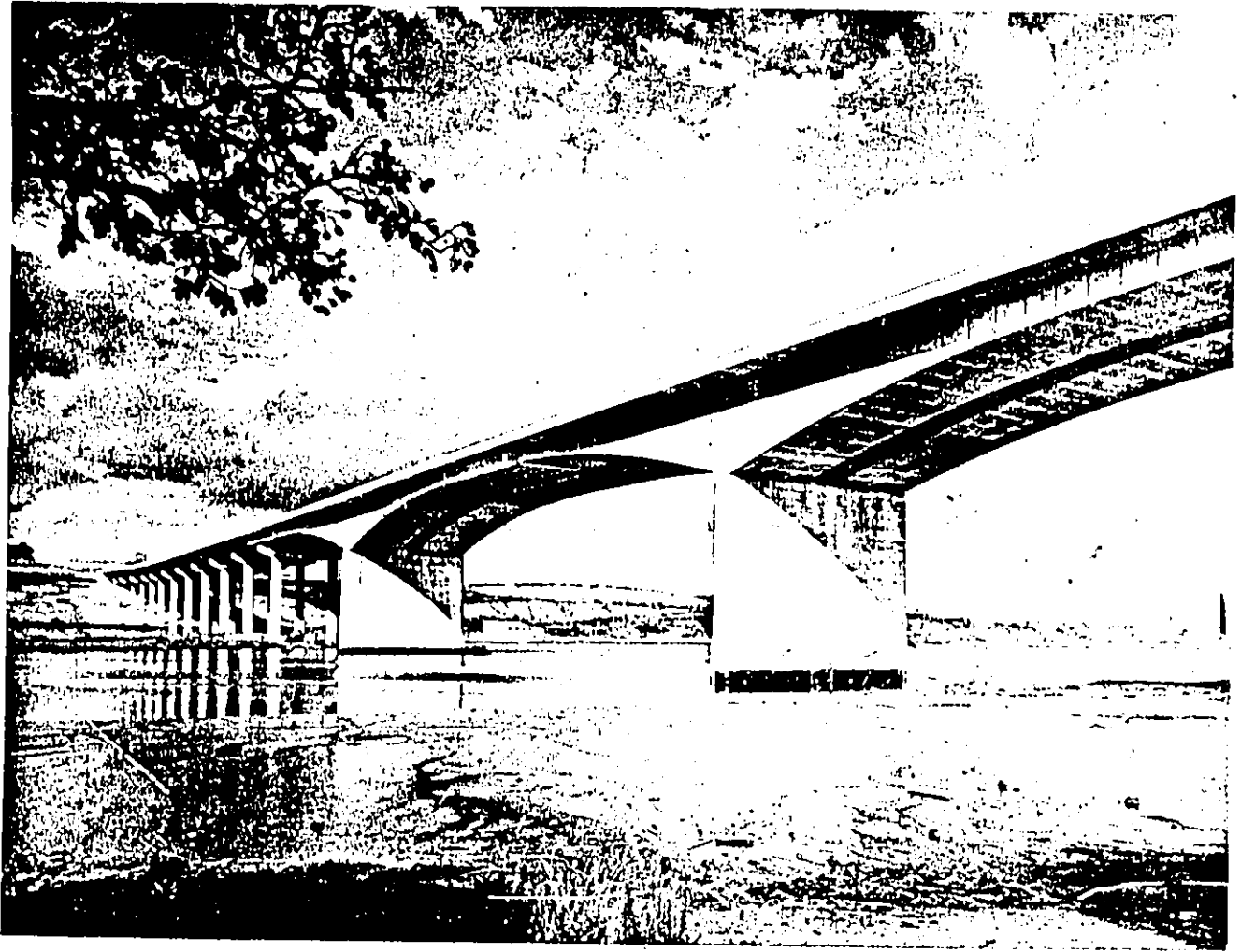


Fig. 7.2 Prestressed Concrete Bridge over the Medway, England.

bridge has two roadways of 7.3 m (23.9 feet) with lateral strips of 0.30 m (1 foot), and shoulders of 2.4 m (7.9 feet) each plus cyclists lanes and sidewalks.

The viaducts consist of prefabricated type I beams for the interior girders and box type for the exterior girders simply supported for the dead load but continuous for the live load. They were erected by a steel launching girder. The bridge over the river was built by the cantilevering method but as the corbels of the hammerhead were non-symmetrical (61.0 m (200.1 feet) the one towards the river and 92.25 m (302.7 feet) the other) temporary piers were used to stabilize the structure (Fig. 7.3). The cross section is a three-cell box girder (four webs with two overhangs) of a variable height (10.73 m (35.2 feet) over the pier and 2.74 m (9.0 feet) at the ends of the corbels).

The suspended span is composed of the same prefabricated beams used for the viaducts and were placed with the same launching girder.

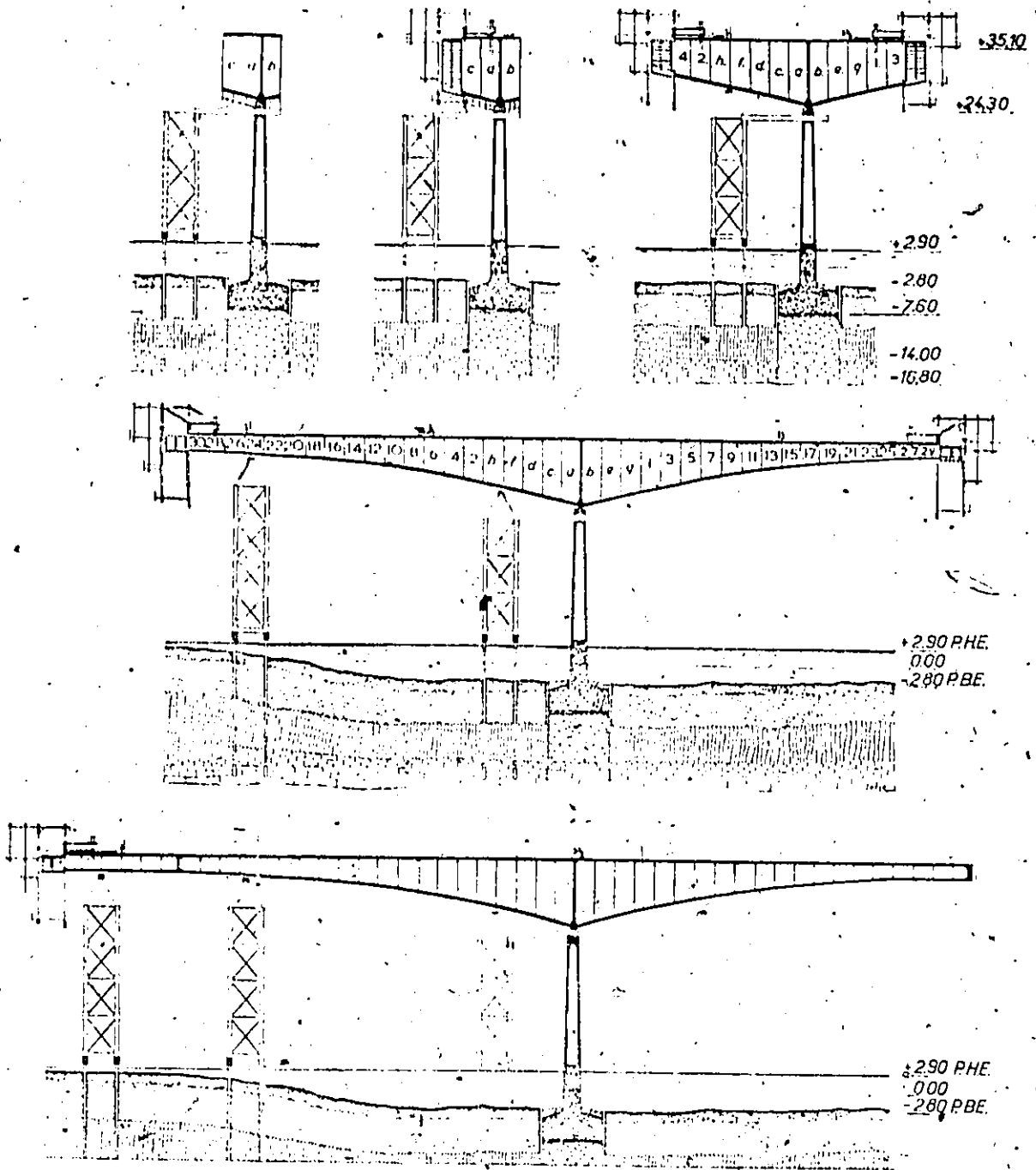


Fig. 7.3 Construction Phases of the Medway Bridge.

7.3 Knight Street Bridge in Vancouver, B.C.
(Ref. 7.3, 7.4)

The main span of this bridge (Fig. 7.4) was opened to traffic in January 1974. It is of the classical type (for the South branch) with two cantilevers and a suspended span which in the original drawings consisted of AASHO type girders.

The particular feature of this bridge is the method used in erecting this suspended span as proposed as an alternative after the bids by the company Dywidag Canada Ltd., the sub-contractor for the prestressing.

This method consisted of continuing the erection of the deck in the cantilever method even for the suspended portion and replacing the AASHO girders by the same section as for the cantilevers (a simple box) and thus avoiding the use of a special launching girder and a discontinuity in the profile of the cantilevers and the drop-in span.

After the cantilever extremities are finished, the seating for the drop-in span is realized and the bearings placed, then the cantilever method process continues (Fig. 7.4) to complete the construction of the bridge. This was made possible by temporarily placing prestressed steel (here type

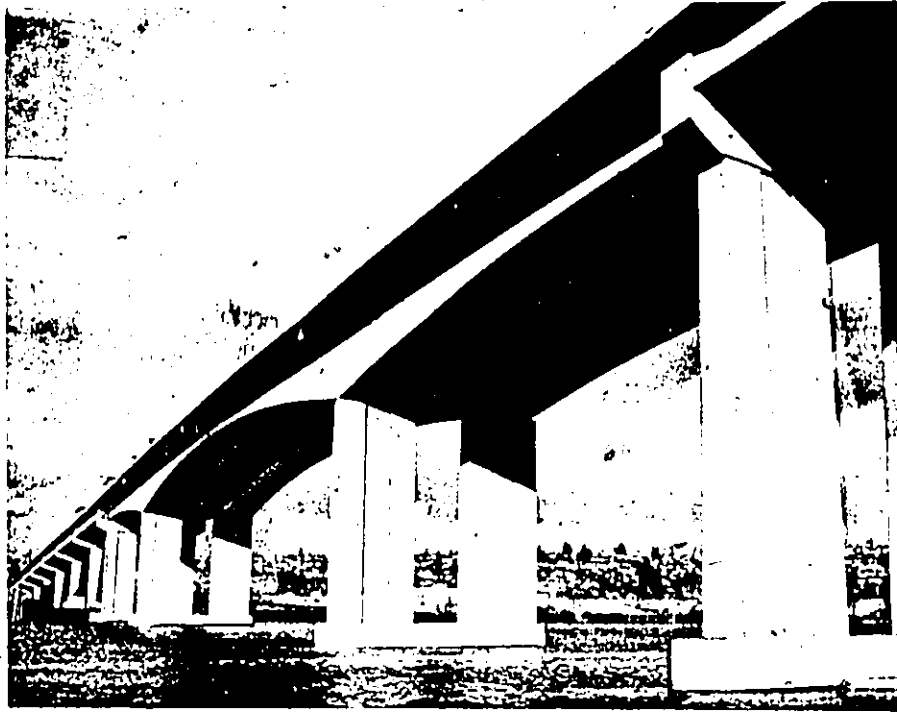
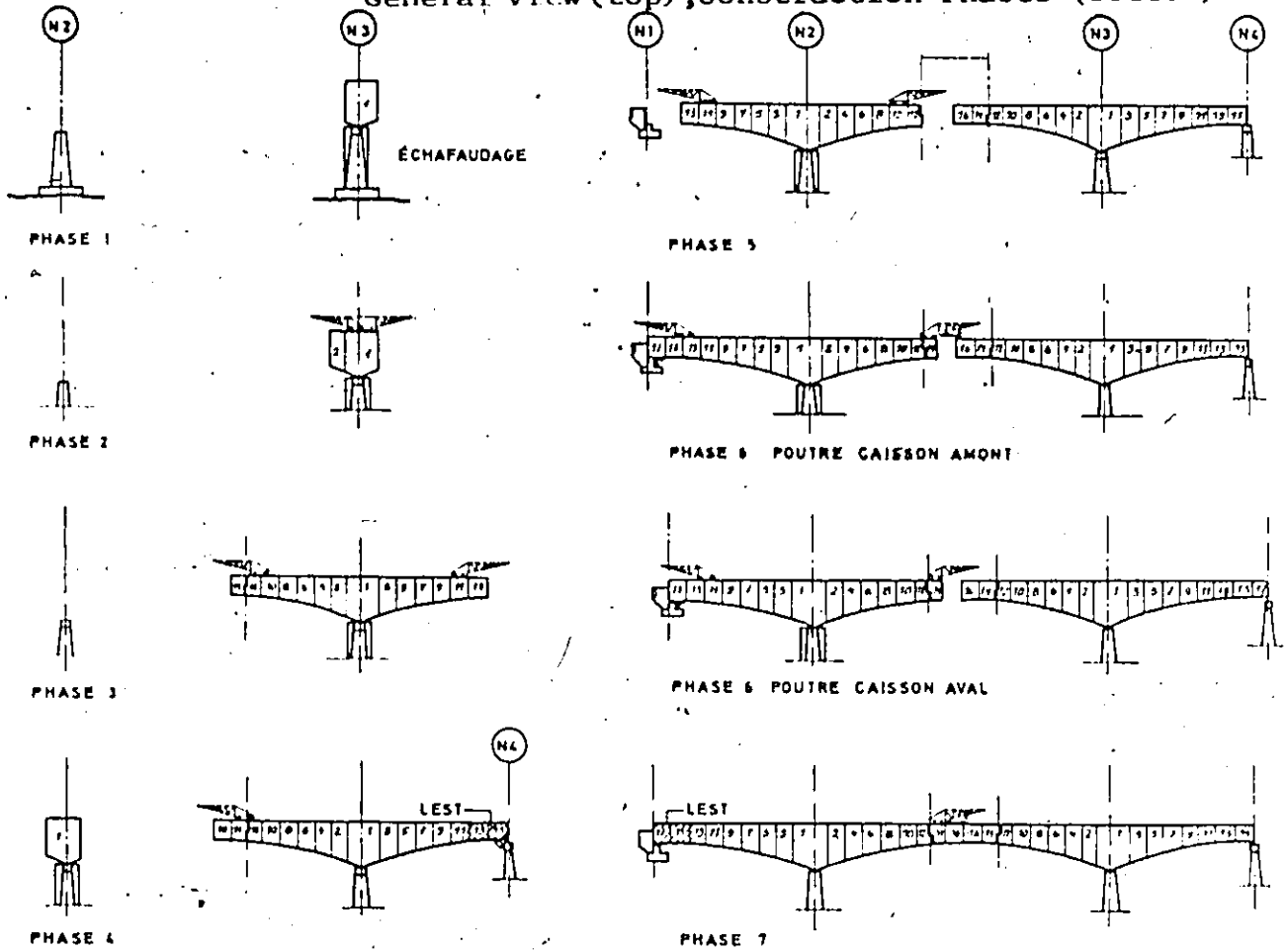


Fig. 7.4 Knight Street Bridge, Vancouver, B.C.

General View (top), Construction Phases (bottom)



Dywidag bars) in the top slab and continuing across the joint between the cantilever and the suspended span. The temporary top steel was cut once the structure was completed.

7.4 Oleron Viaduct (Ref. 7.5 to 7.7)

This viaduct, started in May 1964 and opened to traffic in June 1966, is among the first bridges built by the cantilevering method with segments erected using a launching gantry.

With a total length of 2,862 m (9,389.7 feet) (Fig. 7.5), it has 46 spans (28.75, 7 x 39.50, 59.25, 26 x 79.0, 59.25, 9 x 39.5, 28.75 m) and it is divided in nine sections (394.05, 230.4, 5 x 316, 342.5, 315.05) by expansion joints located approximately at the quarter point of the spans.

The section is a simple box (Fig. 7.6), prestressed longitudinally and transversely by the Freyssinet system. The top slab has a width of 10.6 m (34.8 feet) and a minimum thickness of 20 cm (7.87 inches). The deck is formed by 860 segments prefabricated on the site and transported to the erection point on trolleys travelling on rails along the

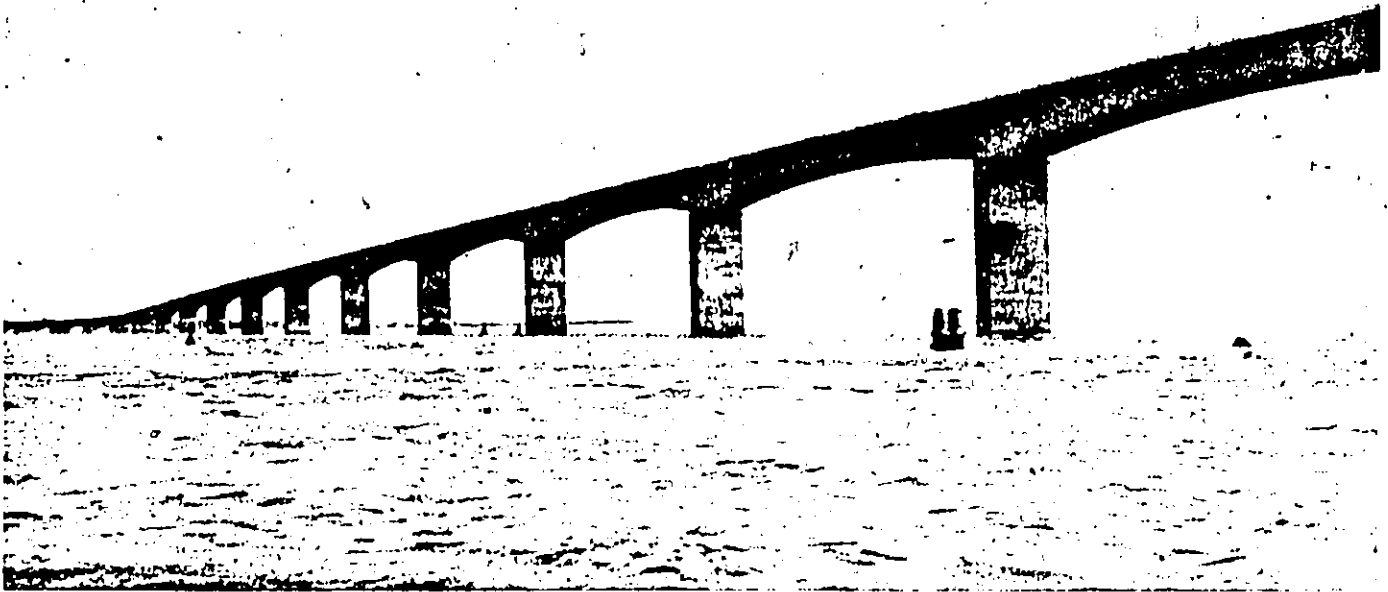


Fig. 7.5 General View of Oleron Viaduct.

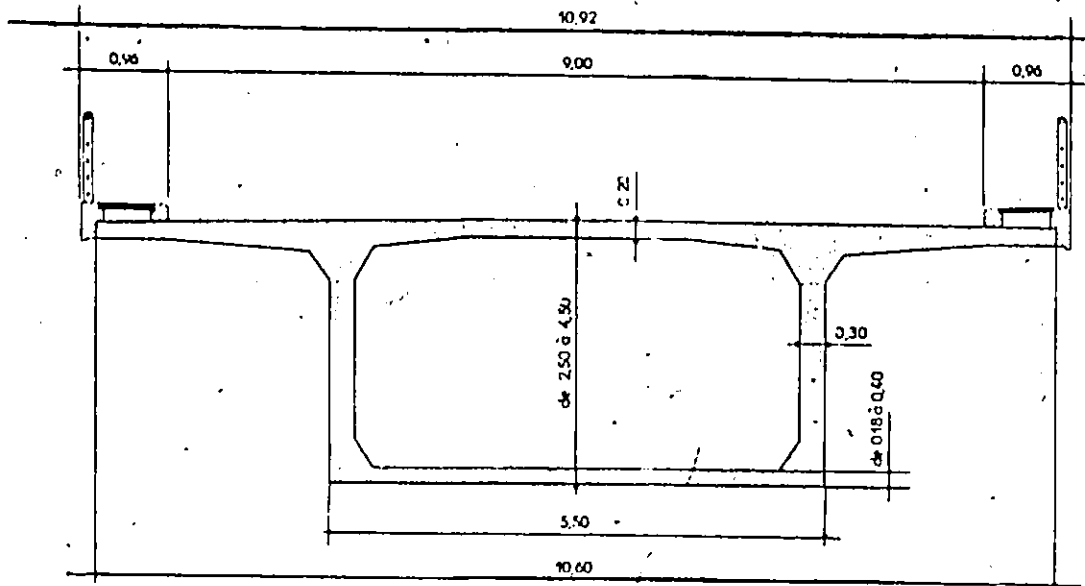


Fig. 7.6 Cross Section of Oleron Viaduct.

already finished deck, thus avoiding the use of any floating equipment which would have been difficult due to the high tides.

The launching gantry is a steel beam (Fig. 7.7) of about 100 m (328 feet) long carried on two legs, one located at the rear and the other in the middle. A hinged prop is attached at the front end. These two legs are designed to allow the free longitudinal passage of the segments transported by a suspended crab hung from the bottom flange of the beam.

This crab allows the segments to be shifted sideways, raised vertically or turned 90° (Fig. 7.8).

The successive phases of the erection operations as described in the article of Muller and Dufoix (Ref. 7.6) are the following:

- (i) Once a span has been completed, the positioning boom is moved forward on buggies provided under the rear and middle legs, until the latter reach the end of a finished hammerhead (Fig. 7.8).
- (ii) The geometry is such that the front support is then located slightly forward of the next pier and can

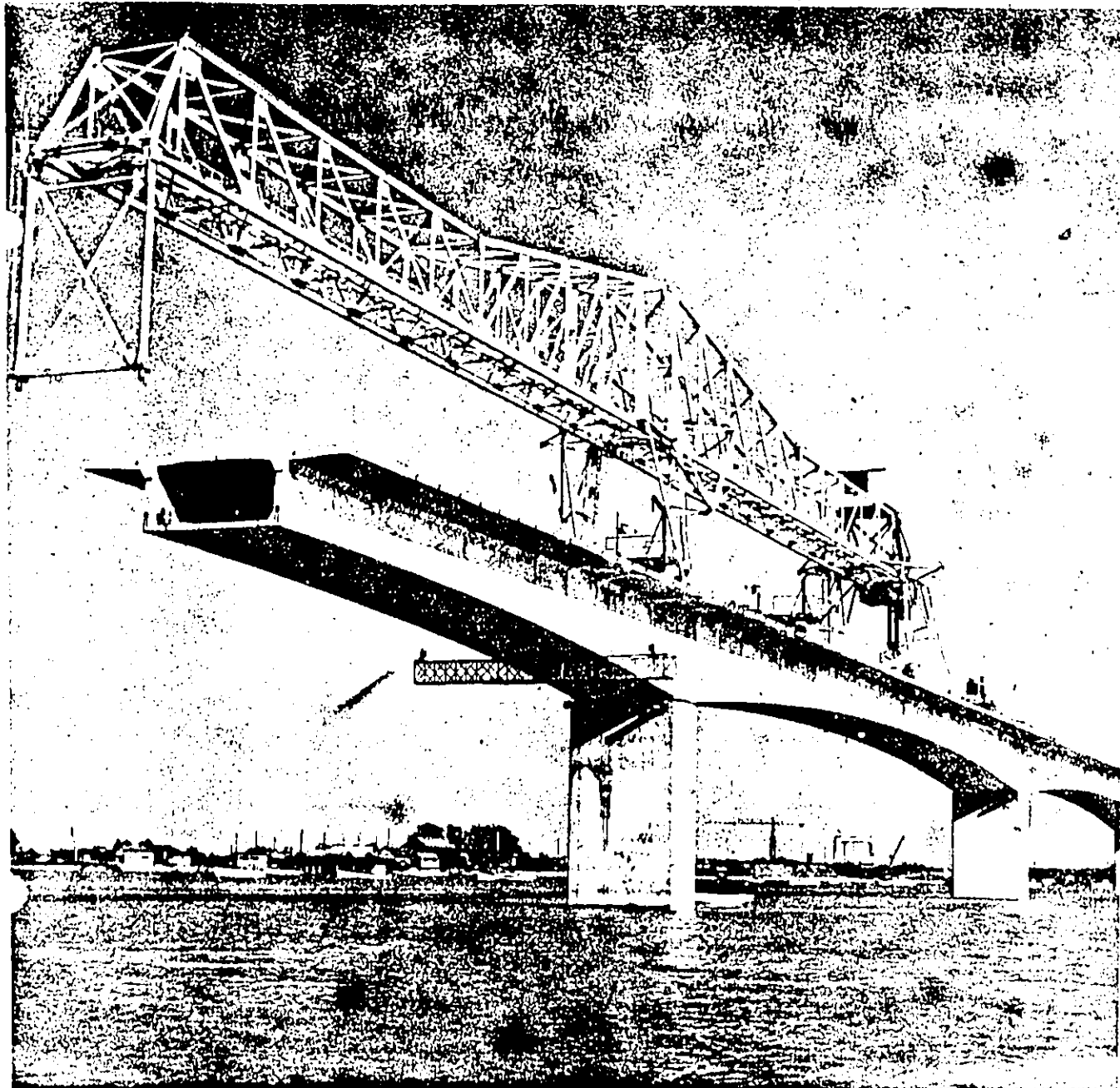
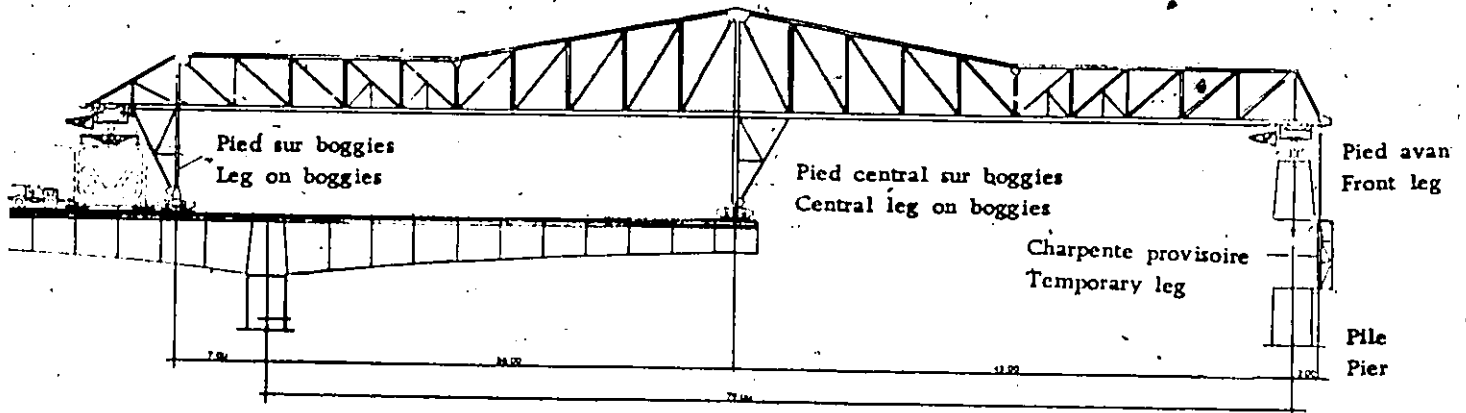
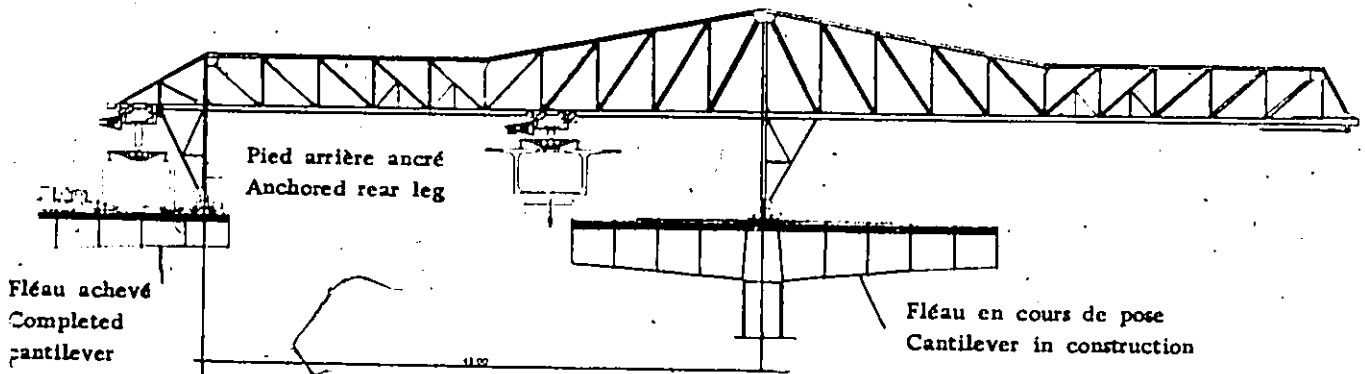


Fig. 7.7 Oleron Viaduct, General View of the Launching Gantry.



1. POSE DU VOUSOIR SUR PILE
PLACING OF VOUSOIR ON PIER



2. POSE DES VOUSOIRS COURANTS
PLACING OF NORMAL VOUSOIRS

Fig. 7.7 Launching Gantry.

be propped on a temporary light frame secured to the pier lining leaving the upper face of the header beam clear. The jacks may be adjusted to ensure optimum weight distribution of the portal crane over the three-point suspension, the system then functioning as a continuous beam.

(iii) The first concrete segment is then fed over the already completed part of the structure, is manoeuvred by the positioning crane and brought forward to the pier header beam.

(iv) A secondary frame, known as a "gin pole", is next positioned over the initial or start segment, and the adjustable jacks are manipulated to transfer the reaction from the front legs to this support. Next, the middle legs are lifted off the first hammerhead, and the launching beam is supported at two ends only.

(v) The positioning frame is now rolled forward until the middle legs are plumb with the pier centerline. The jacks are manipulated in a reverse direction

until the weight of the portal crane is transferred from the gin pole to the middle legs.

- (vi) The suspension crab is used to move back the gin pole, during which time the temporary frame which carried the front legs is dismantled. Cantilever construction proper may now begin.
- (vii) The standard deck segments which are fed over the already completed portion of the bridge, are installed symmetrically on either side of the pier. Consolidating tendons are put under tension at each phase of installation to provide flexural strength of the hammerhead. The operation is repeated until the new hammerhead reaches the end of the preceding one.
- (viii) A keying segment - actually identical with a standard segment - is positioned between the two ends of the hammerheads. Now the final post-tensioning is carried out to consolidate the structure, after which the positioning frame may be rolled forward to the next phase and the cycle

of operations begun anew.

7.5 Grand'Mère Bridge, Province of Québec
(Under Construction) (Fig. 7.9)

This prestressed concrete bridge, analysed and designed by the author, is erected by the cantilever method, and it will have the longest span in North America for this category of bridges (1977).

The central span of 595 feet (181.3 m) is framed on each side by spans of 130 feet (39.6 m) and a triangular nose of 40 feet (12.2 m) long of solid concrete for a total length of 935 feet (285 m).

The side spans are 80% filled with ballast which together with the noses serve to stabilize the cantilevers during construction and prevent any uplift on the exterior supports after the bridge is finished.

The cross section is a simple box with two webs (14 inches (35.5 cm) thick for the central span and 24 inches (61 cm) for the side spans) and a depth varying from 28 feet (8.5 m) to 32 feet (9.7 m) for the side spans, and a variable depth of 32 feet (9.7 m) to 9.5 feet (2.9 m) for the main span (Fig. 7.10).

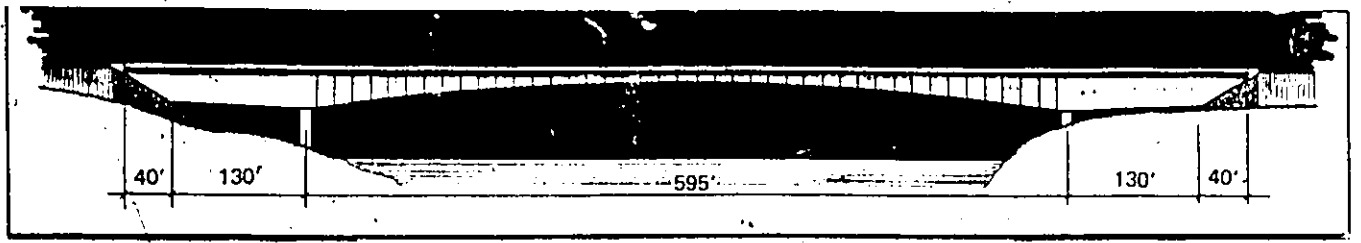


Fig. 7.9 Grand'Mère Bridge.

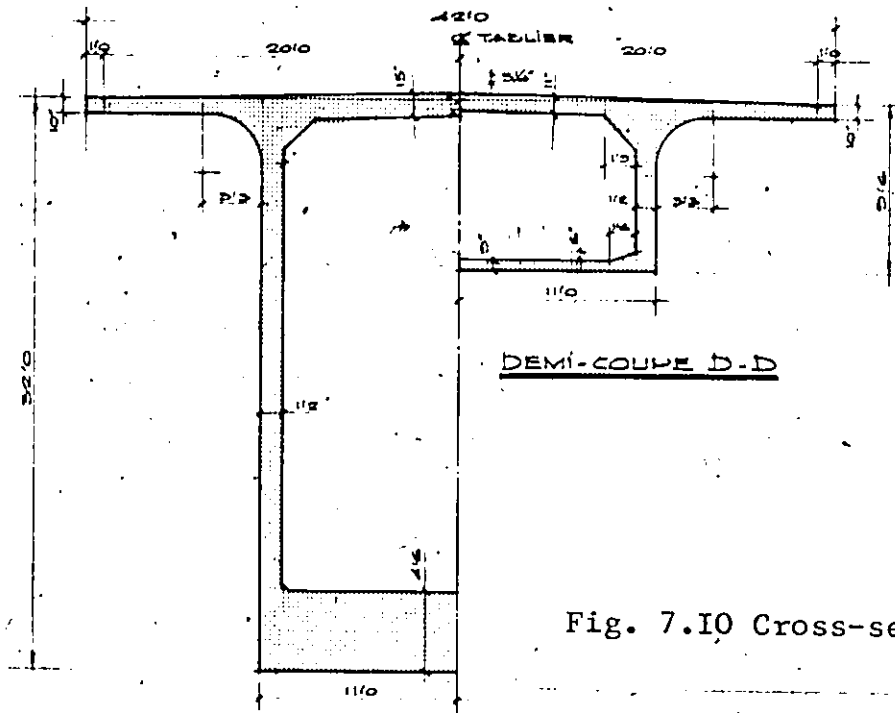
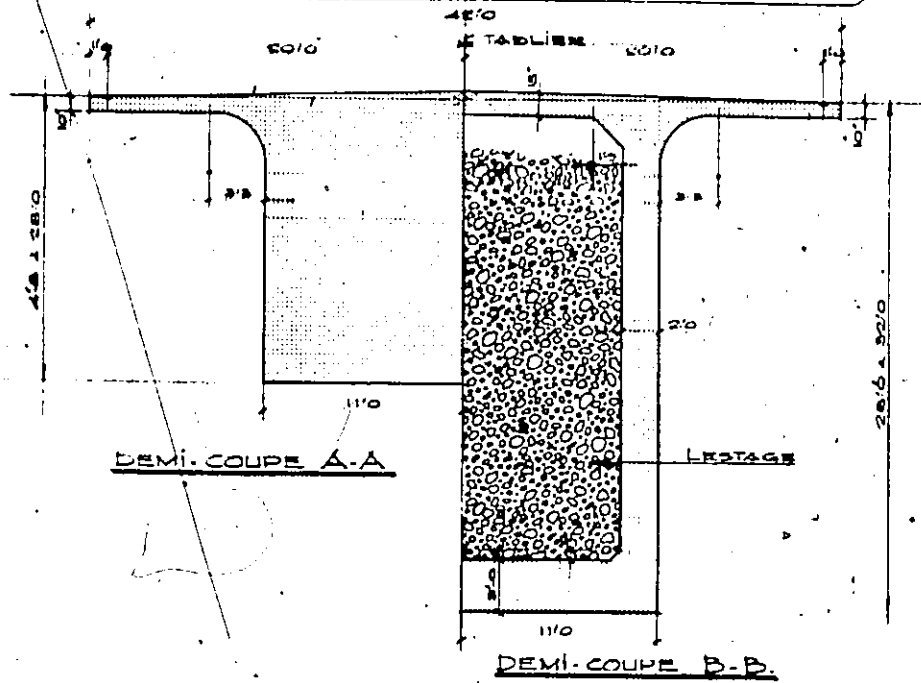


Fig. 7.10 Cross-section.



The top slab of 42 feet (12.8 m) width has two cantilevers varying in thickness from 14 inches (35.5 cm) to 10 inches (25.4 cm). The portion between the webs is thicker at the pier (15 inches, 38.1 cm) and thinner at the center of the main span (11 inches, 28 cm) as the number of prestressing steel bars required is reduced. The bottom slab thickness varies from 9.5 inches (24.1 cm) at the center of the main span to 54 inches (1.37 m) over the piers and remains constant for the side spans at 36 inches (0.91 m).

The deck is prestressed longitudinally, transversely and vertically (for shear) with Dywidag bars of $1\frac{1}{2}$ inches diameter. The side spans and a small portion (9 feet, 2.74 m) of the main span were cast on formwork or on a sandfill of 4 feet (1.22 m) to 5 feet (1.52 m) thick. The main span was erected in the cantilever method with cast-in-place segments of 10 to 15 feet (3.05 to 4.57 m) long using two form-travellers (Fig. 7.11) supplied by the prestressing subcontractor, Dywidag Canada Ltd.

Consulting Engineer: Francis Boulva & Partners Ltd.

Site Supervision: Lalonde, Girouard, Letendre
& Associates, Consulting Engineers

General Contractor: Alta Construction

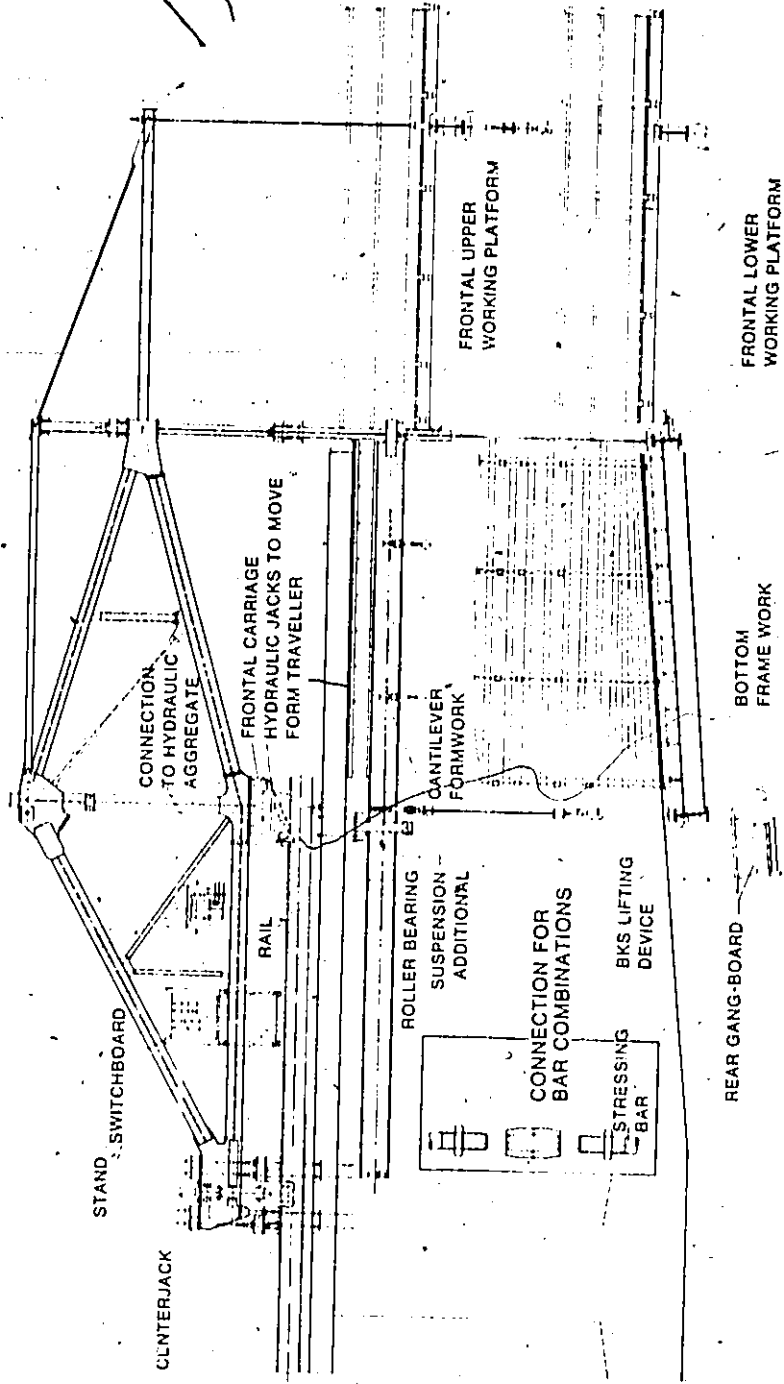


Fig. 7.11 Details of the Form-traveller used at Grand-Mère Bridge.

Post-Tensioning
Sub-Contractor:

Dywidag Canada Ltd.

7.6 Elevated Highway "Mancunian Way" in Manchester,
Great Britain (Fig. 7.12, Ref. 7.8)

This highway, inaugurated in March 1967, is not a typical example of cantilever construction but it was included for the originality of the erection method used for the segments.

With a total length of about 970 m (3,182.4 feet), it has 28 spans of 32 m (105 feet), two spans of 18 m (59 feet) and two end spans of 24 m (78.7 feet). The cross section for each direction of traffic is a simple box with corbels on each side. The two boxes forming the total width are independent during construction but continuous, transversely, after a central mall serving as a longitudinal beam is poured.

All the segments were prefabricated and dimensioned so as to use only mobile cranes for the handling and erection.

The originality of the erection is that the segments were placed one after the other on an auxiliary beam suspended by the finished part of the deck (Fig. 7.13). Instead of

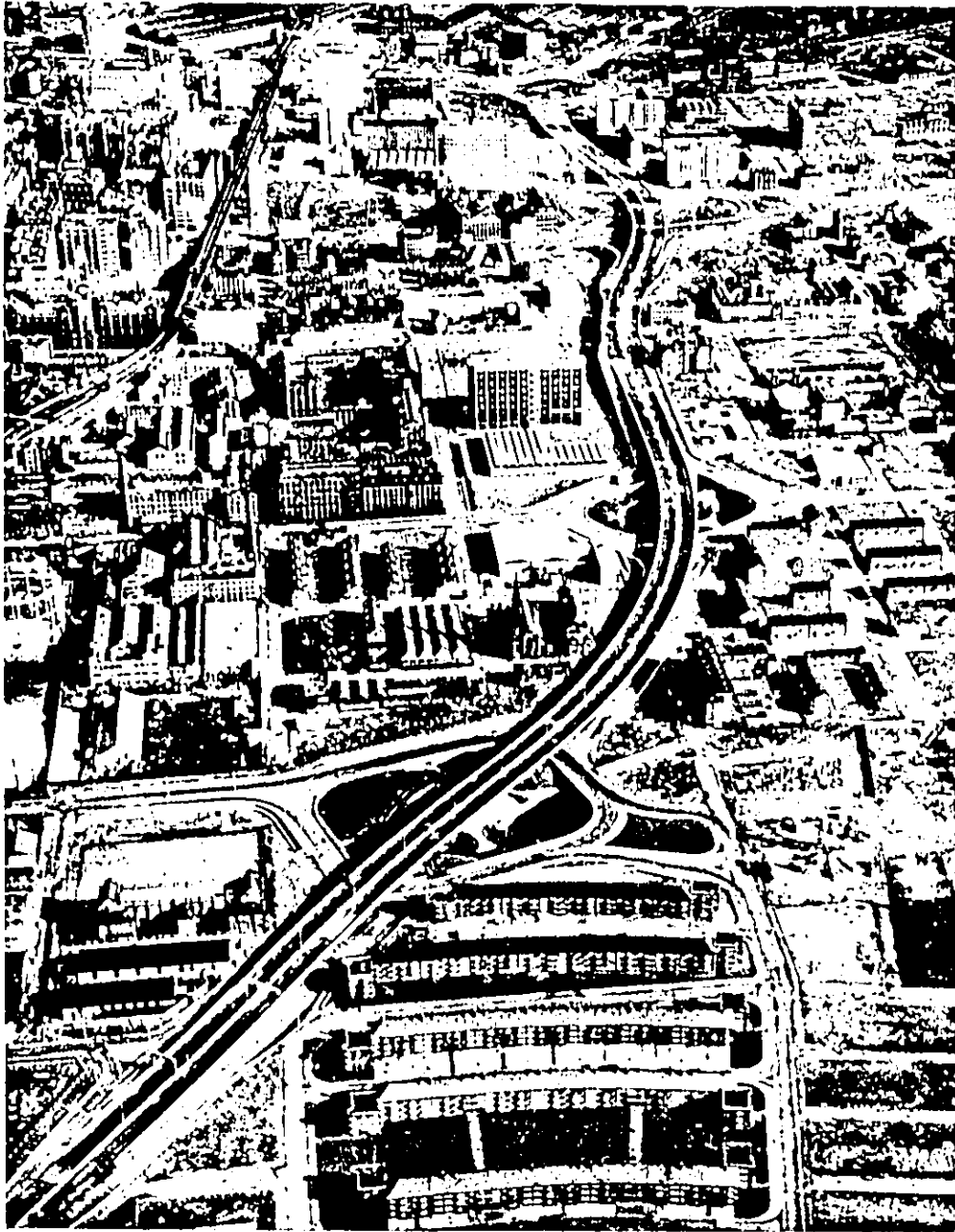
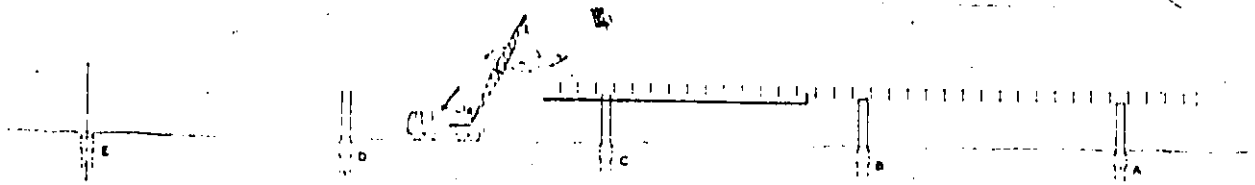


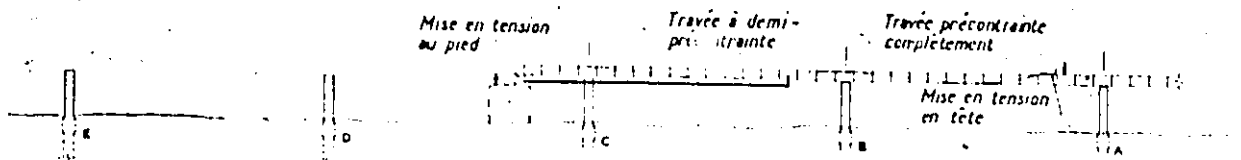
Fig. 7.12 Mancunian Way.



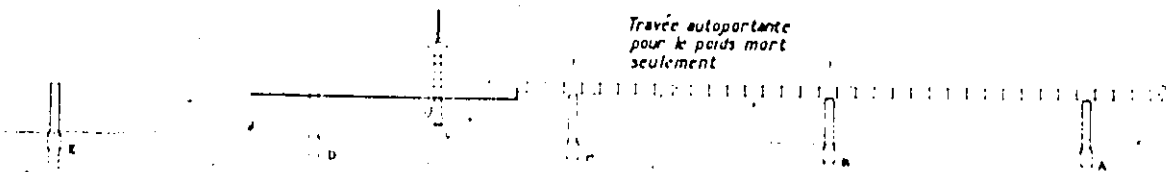
Phase I. Placing the Scaffolding



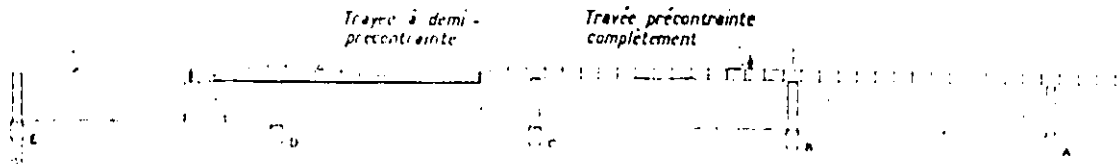
Phase 2. Placing the Prefabricated Segments..



Phase 3. First Prestressing.



Phase 4. Moving the scaffolding
Placing more segments.



Phase 5. Second Prestressing.

Fig. 7.I3 Construction Phases of Mancunian Way.

using cast match joints, the segments were placed with a 75 mm (2.95 inches) gap which was filled with grout once all the segments were placed.

The prestressing steel, placed in the box webs, was comprised of Freyssinet cables, 12 strands of 15 mm, and their layout was similar to the one used for a structure cast on scaffolding. Each cable covered two spans and was anchored at one end to the top slab and at the other end to the exterior face of the bottom slab.

Once all the segments of one span were placed and their joints grouted, half of the necessary cables were stressed thus insuring the structural capacity of the new span for its own weight, while for the previous span these cables provided its final capacity since these cables were the second series to be stressed (final prestressing).

CHAPTER 8

CONCLUSIONS

As already mentioned there has been a growing interest in North America in recent years on this type of structure.

This interest has already given some good results such as Knight Street Bridge in Vancouver and Pine Valley Creek Bridge in California amongst others. A lot more are on the drafting tables or under construction throughout this continent and even the pushing method (see paragraph 5.6) is being used in Indiana (Fig. 5.48).

An indication of the importance given by the United States and Canada to this type of structure is the number of PCI-PTI committees which were formed to gather and review all available information pertaining to segmental construction of structures. Some recommendations have already been published (Ref. 5.10, 6.16). Even tests on 1 to 10 scale models have been made by the Portland Cement Association for the Three Sisters Bridge (Ref. 8.1).

The interest manifested in this method is understandable. The simplicity of erection, the moderate investments, the adaptability of the method to any kind of shape of bridge and the control of the finished product are amongst the benefits of the method.

Although it is now close to thirty years since the segmental erection of concrete bridges was first used, the method remains new and under continuous improvement and modification.

Some examples are the use of multiple keys for the prefabricated segments (Fig. 5.34) or the reduction of the dead load by using ribbed thin webs (Meules Bridge, Fig. 3.6).⁴

More research has to be made in the use of light weight concrete which would allow longer spans and conservation of materials.

Improvement can also be made in the erection techniques to speed up the placing of segments and reducing the construction time, minimize working hours and lower the construction costs. But all these improvements can only be possible by the close collaboration of all the interested parties, that is, the owner, the consulting engineer and the general contractor.

Special seminars also have to be held by organizations such as the Prestressed Concrete Institute or the Portland Cement Association to familiarize engineers with this kind of structure because as already mentioned the method is still very new in our country and our neighbours to the South.

A bigger participation of the Engineering Faculties of the Universities is also desirable. In fact a lot of problems can be solved for the Engineer by the use of tests on scale models with a great benefit, in the same time, for the students.

In ending, the author hopes that he has contributed a little to the reduction of the mystique of these bridges.

REFERENCES

CHAPTER 1.

- 1.1 C. Carlson & N. Ady,
"Segmented Posttensioned Construction."
Special Bibliography, #185,
Portland Cement Association, Skokie, Ill. March 1968.
- 1.2 G.C. Lacey, J.E. Breen, N.H. Burns,
"State for the Art for Long Span Prestressed
Concrete Bridges of Segmental Construction."
P.C.I. Journal, September-October 1971.

CHAPTER 2.

- 2.1 Derrick Beckett,
"Great Buildings of the World, Bridges".
Paul Hamlyn, London 1969.
- 2.2 Jean Muller,
"Long-Span Precast Prestressed Concrete Bridges
Built in Cantilever".
ACI Publication SP 23 "First International Symposium,
Concrete Bridge Design" Detroit 1967.
- 2.3 Jean Laborie,
"The Prestressed Concrete Bridges at Worms Over
the Rhine and Coblenz Over the Mosel".
La Technique des Travaux, Liège (in french),
March-April 1956

CHAPTER 3.

- 3.1 Anonymous
"Cantilever Construction of Bridges".
Freyssinet International, 1975
- 3.2 Anonymous
"Dywidag International - A Leading Enterprise
in the Building World".
Dyckerhoff & Widmann AG, Munich
- 3.3 J. Courbon,
"The bridge of Pyle".
Annales de l'I.T.B.T.P., In French, Paris December 1968
- 3.4 Anonymous
"Contractors Match Methods to Mountains on
Alpine Route."
Engineering News-Record, July 12, 1973
- 3.5 A. Oeconomou
The Bridge of Tartana in Greece.
Reprint from the Technical Annuals of the
Technical Chamber of Greece, In Greek, April 1971
- 3.6 Anonymous
"Rotterdam Motorway".
Simmast of Canada Limited, Toronto
- 3.7 Anonymous
French Prestressed Concrete Structures.
7th International Congress of F.I.P.,
New York, 1974
- 3.8 J. Mathivat
"The Evolution of Construction Methods for
Prestressed Concrete Bridges".
Reprint from "L'Ingénieur Constructeur",
Special Issue, "Ouvrages d'Art", In French,
Paris, April 1971

CHAPTER 3 CONTINUED

- 3.10 Y. Guyon
The Cantilever Construction of Bridges.
In French, STUP, Paris, 1964

CHAPTER 4

- 4.1 G. Worontzoff
"Prestressed Concrete Bridges Built by the
Cantilever Method".
La Technique des Travaux, In French, March-April 1964.
- 4.2 Anonymous
Cantilever Construction of Bridges.
Freysinet International, Paris, 1971
- 4.3 A. Bouchet
The Prestressed Concrete Bridge Over the Medway
(England). La Technique des Travaux, (In French)
September-October 1964
- 4.4 Pignet, S.A.
The Chillon Viaduct (Switzerland).
La Technique des Travaux, (In French), January-
February 1971
- 4.5 H. Schambeck - M. Bockel
The Cantilevering. A Tested Construction Method for
Prestressed Concrete Bridges.
Anniversary Edition for Ulrich Finsterwalder -
50 Years with Dywidag (In German).
Dyckerhoff & Widmann AG, Munich, 1973

CHAPTER 4. CONTINUED.

- 4.6 Anonymous
Dywidag International.
Dyckerhoff & Widmann AG, In German, Munich.
- 4.7 Anonymous
"Reconstruction of Choisy-le-Roi Bridge".
Annales de l'I.T.B.T.P., In French, December 1964
Paris
- 4.8 H.H. Janssen
Final Design Procedures Following Preliminary
Design.
PCI Seminar on Design of Precast Segmental
Bridges, Chicago, Ill., September 25, 1973
- 4.9 Y. Guyon
"Prestressed Concrete Construction, 2nd Volume".
Eyrolles, In French, Paris, 1968
- 4.10 Anonymous
Cantilever Construction of Bridges
Freyssinet International Company, Paris 1975

CHAPTER 5.

- 5.1 Anonymous
"Cantilever Construction of Bridges".
Freyssinet International, Paris, 1975
- 5.2 H. Schambeck - M. Bockel
The Cantilevering. A Tested Construction Method
for Prestressing Concrete Bridges.
Anniversary Edition for Ulrich Finsterwalder -
50 Years with Dywidag (In German)
Dyckerhoff & Widmann AG, Munich, 1973

CHAPTER 5 CONTINUED.

- 5.3 Anonymous
Bosques de Reforma Viaducts.
Administration Council of Bosques de Los Lomas,
(In Spanish). Mexico, 1972
- 5.4 Anonymous
Dywidag Prestressed Concrete a Great Success
Around the World.
Dyckerhoff & Widmann AG, In English, Munich
- 5.5 Anonymous
"Dywidag Berichte 4-1967.",
Dyckerhoff & Widmann AG, Munich
- 5.6 Anonymous
"Cantilever Erection of Bridges"
STUP, (In French), Paris, July 1963
- 5.7 G. Worontzoff
"Prestressed Concrete Bridges Built by the
Cantilever Method."
La Technique des Travaux, (In French), Liège,
March-April 1964
- 5.8 Anonymous
"Cantilever Erection of Bridges."
STUP, (In French), Paris, July 1963.
- 5.9 Anonymous
Cantilever Construction of Bridges.
STUP, Paris, 1970
- 5.10 PCI Committee on Segmental Construction
"Recommended Practice for Segmental Construction
in Prestressed Concrete".
Journal of PIC, March-April 1975

CHAPTER 5 CONTINUED

- 5.11 J. Muller
"Evolution and Recent Development of the
Cantilever Construction Method."
French Technical Contributions to the 7th
International Congress of the F.I.P.,
(In French) New York, 1974
- 5.12 J. Mathivat
"The Evolution of the Construction Method for
Prestressed Concrete Bridges."
Reprint from "L'Ingénieur Constructeur", Special
Issue, "Ouvrages d'Art", (In French),
Paris, April 1971
- 5.13 Anonymous
"Prestressed Concrete Structures in Italy 1970-1974"
7th International Congress of the F.I.P. Congress,
New York, 1974
- 5.14 F. Leonhardt
"Construction Method of a Prestressed Concrete
Bridge by Alternation of Concreting and Driving."
Annales de l'I.T.B.T.P., (In French), Paris, January 1973
- 5.15 Anonymous
"First Incrementally Launched Post-Tensioned Box
Girder Bridge to be Built in the United States."
Bridge Report of the Post-Tensioning Institute,
December 1976
- 5.16 Anonymous
French Prestressed Concrete Structures.
7th International Congress of the F.I.P.,
New York 1974
- 5.17 Launay, Baum; Chassagnette, Stouft
"Pushing" The New Construction Method of Building
Prestressed Concrete Bridges".
Annales de l'I.T.B.T.P., (In French), Paris, May 1970

CHAPTER 5 CONTINUED

- 5.18 M. Rondet
"Construction of Prestressed Concrete by the
Pushing Method".
Expomat Actualités, (In French), Paris, February 1976
- 5.19 G. Worontzoff
"The Construction of Prestressed Concrete Bridges
by the Cantilever Method."
Reprint from "L'Ingénieur Constructeur", Special
Issue, "Ouvrages d'Art", (In French), April 1971
- 5.20 F. Hugenschmidt
Epoxy Adhesives in Precast Prestressed Concrete
Construction
Journal of the PIC, March-April 1974
- 5.21 Y. Guyon
"Prestressed Concrete Construction, 2nd Volume."
Eyrolles, (In French), Paris 1968.

CHAPTER 6.

- 6.1 J. Muller
"Evolution and Recent Developments of the Cantilever
Construction Method"
French Technical Contribution to the 7th International
Congress of the F.I.P., (In French), New York, 1974
- 6.2 Anonymous
Basques de Reforma Viaducts.
Administration Council of Basques de Las Lomas,
(In Spanish), Mexico, 1972.
- 6.3 Anonymous
"Prestressed Concrete Structures in Italy 1970-1974"
7th International Congress of the F.I.P.,
New York, 1974

CHAPTER 6 CONTINUED

- 6.4 Anonymous
Dywidag International.
Dyckerhoff & Widmann AG, (In German) Munich
- 6.5 J. Mathivat
"Pier Structures adapted to the Cantilever
Construction"
French Technical Contribution to the 7th International
Congress of the F.I.P., (In French), New York, 1974
- 6.6 Libby and Perkins,
"Highway Bridge Superstructure Design Principles and
Construction Methods"
Grantville Publishing Company, San Diego,
California, 1976
- 6.7 Y. Guyon
"Prestressed Concrete Structures, 2nd Volume"
Eyrolles, Paris, 1968
- 6.8 Anonymous
International Recommendations for the Design and
Construction of Concrete Structures.
Springer Verlag, New Yrk, 1973
- 6.8 Anonymous
"Conception and Design of Prestressed Concrete"
Edited by the French Government, Paris, 1973
- 6.9 Anonymous
International Recommendations for the Design and
Construction of Concrete Structures
CEB-FIP, Prague 1970
- 6.10 A.Pucher
Influence Surfaces of Elastic Plates.
Springer Verlag, New York, 1973

CHAPTER 6 CONTINUED

- 6.11 H. Homberg and W. Ropers
Fahrbahnplatten Mit Veranderlicher Dicke.
2 Volumes. (In German)
Springer Verlag, Berlin, 1965
- 6.12 M. Thenoz
"Redistribution of the Stresses in the Cantilever
Erected Bridges, the Designer's point of Vue"
7th International Congress of the F.I.P., (In French)
New York, 1974.
- 6.13 R. Atallah & M.Y. Lau
"Redistribution of the Stresses in the Cantilever
Erected Bridges, The Laboratory's Point of Vue"
7th International Congress of the F.I.P., (In French)
New York, 1974.
- 6.14 L. Pliskin
"Design of Croix de Pierre Bridge. Computation of the
Deflection During Construction"
Europe Etudes, (In French), Paris, June 1967
- 6.15 B. Ernari Diaz
"The Technique of Glueing Precast Elements of
Bridges Built by the Cantilever Method"
Technical Publication #52 of Antonio A. Noronha,
Consulting Engineers, Rio de Janeiro, 1974
- 6.16 PCI Bridge Committee
"Tentative Design and Construction Specification
for Precast Segmental Box Girder Bridges"
Journal of the PCI, Vol. 20, No. 4, July-August 1975
- 6.17 J. Muller
"Ten Years of Experience in Precast Construction"
PCI Journal, Vol. 20, No.1, January-February 1975,
1128-61

CHAPTER 6 CONTINUED

- 6.18 Dyckerhoff and Widmann "Creep & Shrinkage".
Technical Publication No. 6
Dyckerhoff & Widmann, (In German), Munich, February 1970
- 6.19 Dyckerhoff and Widmann "Camber Calculation".
Technical Publication No. 9
Dyckerhoff & Widmann, (In German), Munich, September 1970
- 6.20 K.G. Tamberg and P.T. Mikluchin
"Torsional Phenomena Analysis and Concrete Structure
Design"
American Concrete Institute publication SP-35, Detroit 1973

CHAPTER 7

- 7.1 A. Bouchet
"Bendorf Prestressed Concrete Bridge Over the Rhine"
La Technique des Travaux, (In French)
September-October 1966
- 7.2 A. Bouchet
"Prestressed Concrete Bridge Over the Medway"
(England)
La Technique des Travaux, (In French)
September-October 1964
- 7.3 I. Toth
"Knight Street Bridge in Vancouver, B.C., Canada"
La Technique des Travaux, (In French)
March-April 1973
- 7.4 H.K. Aschenbroich
"Cast-in-Place Bridge Segments Save \$200,000"
ACI Journal, March 1974
- 7.5 A. Dufoix
"The Oleron Island Viaduct"
La Technique des Travaux. (In French)
January-February 1967

CHAPTER 7 CONTINUED

- 7.6 J. Muller and A. Dufoix
"The Oleron Island Viaduct"
"Cantilever Construction of the Bridge,
STUP, Paris, 1970"
- 7.7 Anonymous
"European Prestress Specialists Perfect their
Bridge Technique"
Engineering News-Record, October 14, 1965
- 7.8 A. Bouchet
"Mancunlan Way in Manchester, England Urban
Elevated Highway in Prestressed Concrete"
La Technique des Travaux; (In French)
March-April 1968

CHAPTER 8

- 8.1 W.G. Corley, H.G. Russell, A.E. Cardenas,
J.M. Hanson, J.E. Carpenter, N.W. Hansen,
T. Helgason, E. Hognestad
"Design Ultimate Load Test of 1/10-Scale
Micro-Concrete Model of New Potomac River
Crossing, I-266"
PCA Research and Development Bulletin RD011
- R

APPENDIX A

A simple approach to understand the effect of creep upon the modified structure is represented on Fig. A1.

Open System

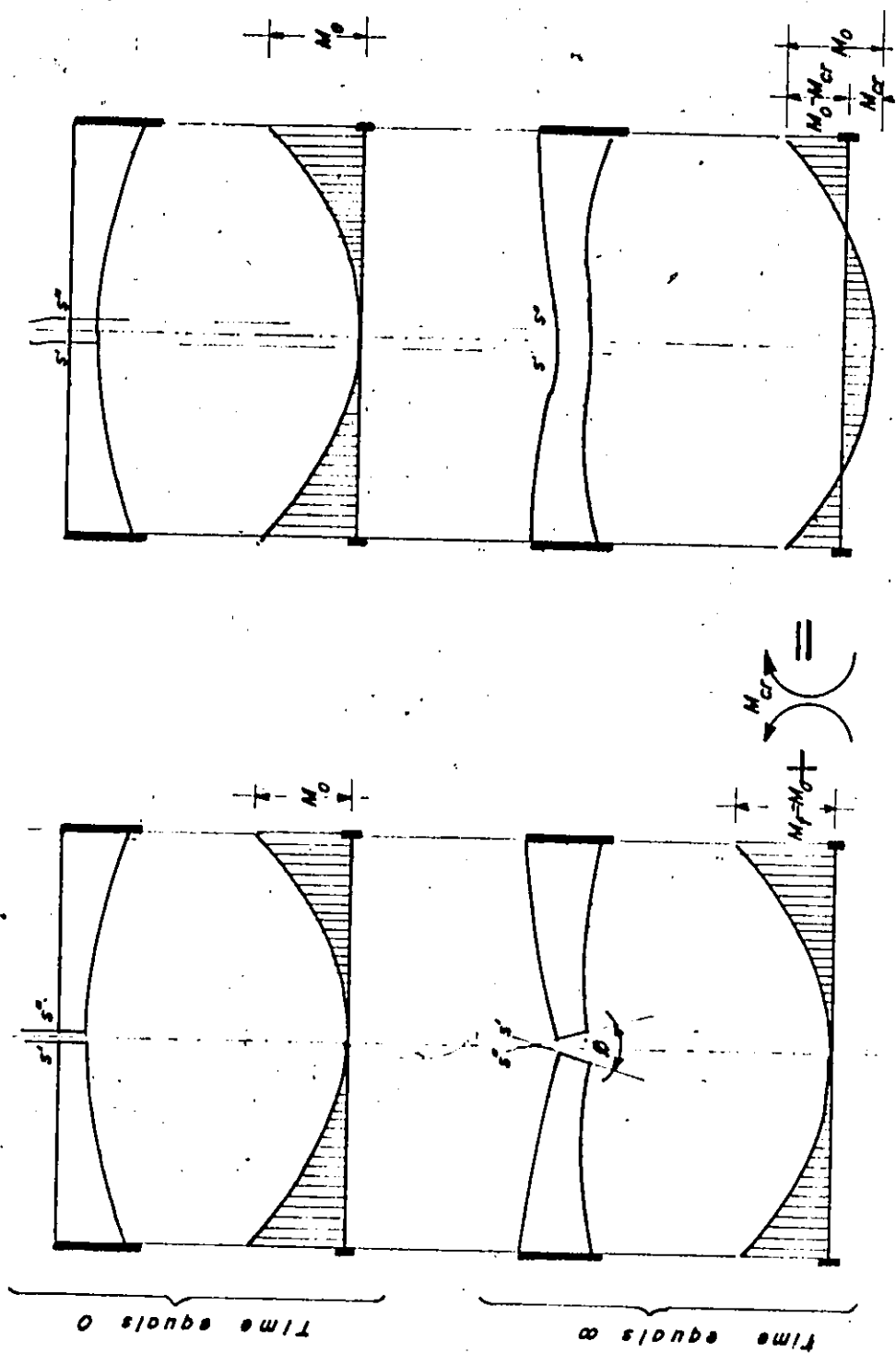
At time equal zero that is immediately after the two cantilevers are finished their end cross sections s' , s'' are parallel and negative bending moment exists throughout the structure.

At time approaching infinity due to the plastic deformation of the concrete these two cross sections will form an angle of \emptyset between them.

This deformation is due to the creep of the concrete and it is realized without any change of the section loads.

Closed (Continuous) System

The cross sections parallel at the end of the construction have to remain parallel even at time approaching infinity due to the closing pouring of the gap between the cantilevers.



a) NO MODIFICATION OF THE STATICAL SYSTEM AFTER CONSTRUCTION

b) CONTINUOUS SYSTEM AFTER CONSTRUCTION

Fig. A1 Bridge profile and bending moments at different times for cantilever erected spans.

In order to realize this a positive moment has to be applied on the open system to turn the two sections through an angle $\phi / 2$ each.

APPENDIX B

ULTIMATE CAPACITY OF A BOX GIRDER

VERSUS AN OPEN SECTION

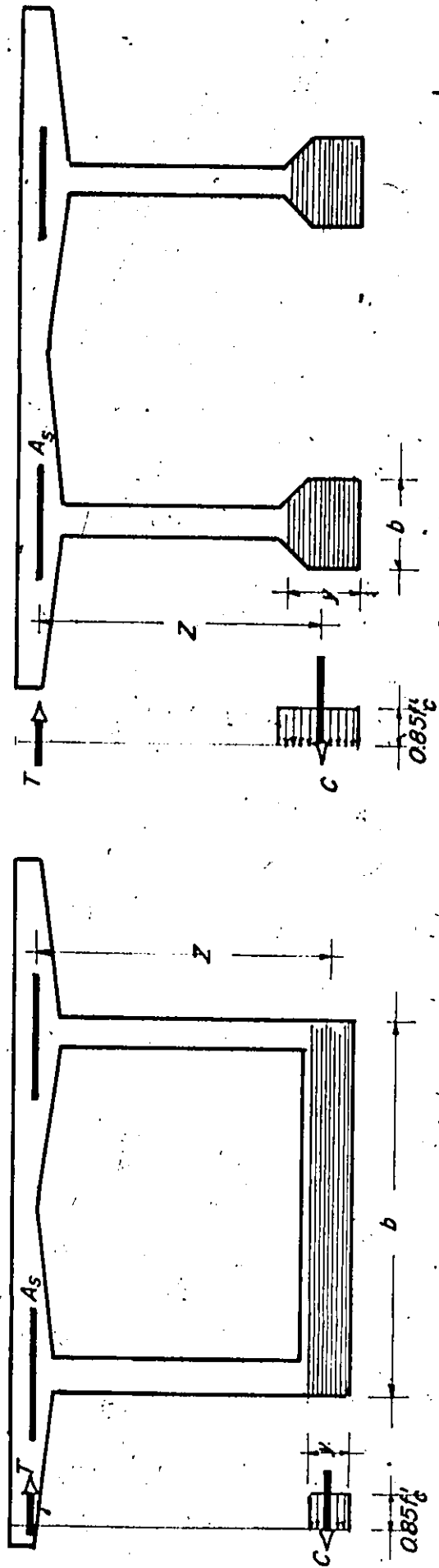
During the failure of the section the resultant of the compression stresses C in the solid concrete under the cracks is equal to the yield force T of the steel (Fig. B1).

$$C = T = 0.85 y b f'_c$$

From this expression it is evident that as T is a given value if the width b of the cross section is very large as in a box section, a reduction of the concrete strength f'_c will affect very little the weight of the compression prism.

On the contrary, if b is small as in an open section any reduction of the value of a f'_c will increase the value of y which will result in decreasing the value of the lever arm Z between the forces C and T .

But as the ultimate moment is equal to ZT it is evident that as Z is reduced the ultimate moment M_{ult} will be reduced.



$$C = T = 0.85 \cdot y \cdot b \cdot f'_c \quad y = T / 0.85 \cdot b \cdot f'_c$$

$$M_u = T \cdot Z$$

Fig. BI Ultimate Capacity of a Box Girder versus an open Section.