Kuwait's Bubiyan Bridge — a 3-D Precast Segmental Space Frame

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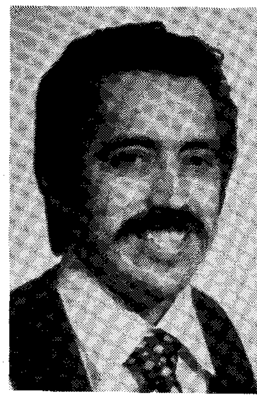


SYNOPSIS

A new and novel precast prestressed segmental concrete bridge is nearing completion between the mainland of Kuwait and the Island of Bubiyan. The superstructure is an externally post-tensioned concrete segmental three-dimensional space frame truss.

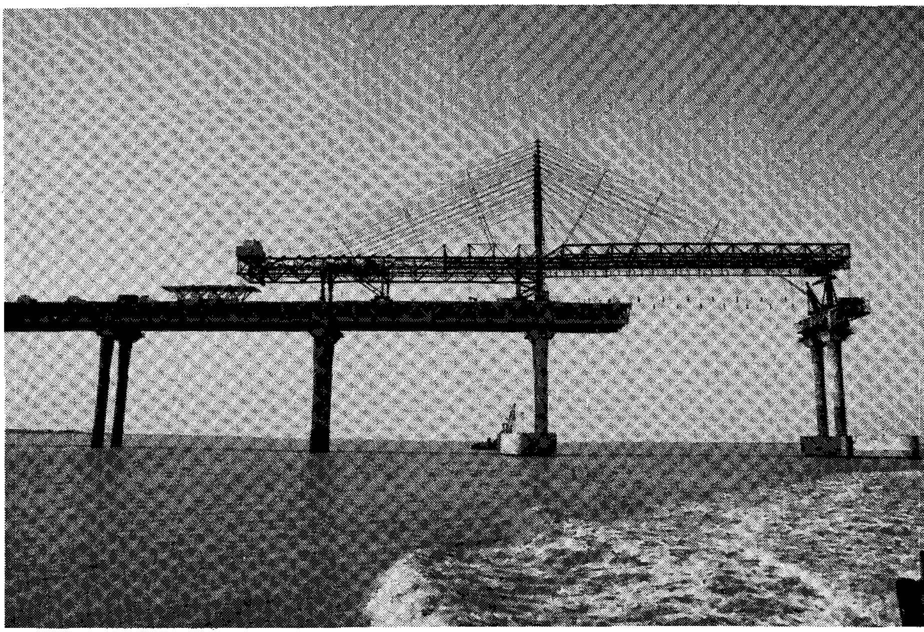
This paper documents the significant aspects of the project development; review and evaluation of turnkey proposals; design, fabrication and erection of the superstructure; and model testing of the superstructure.

The Bubiyan Bridge project represents a significant step forward in the state-of-the-art of precast prestressed segmental bridge construction.

Kuwait State is located at the northwest corner of the Arabian Gulf (Fig. 1). It is bounded on the north and west by Iraq, on the south by Saudi Arabia, and on the east by the Arabian Gulf. The State of Kuwait also includes several offshore islands, of which Bubiyan is the largest.

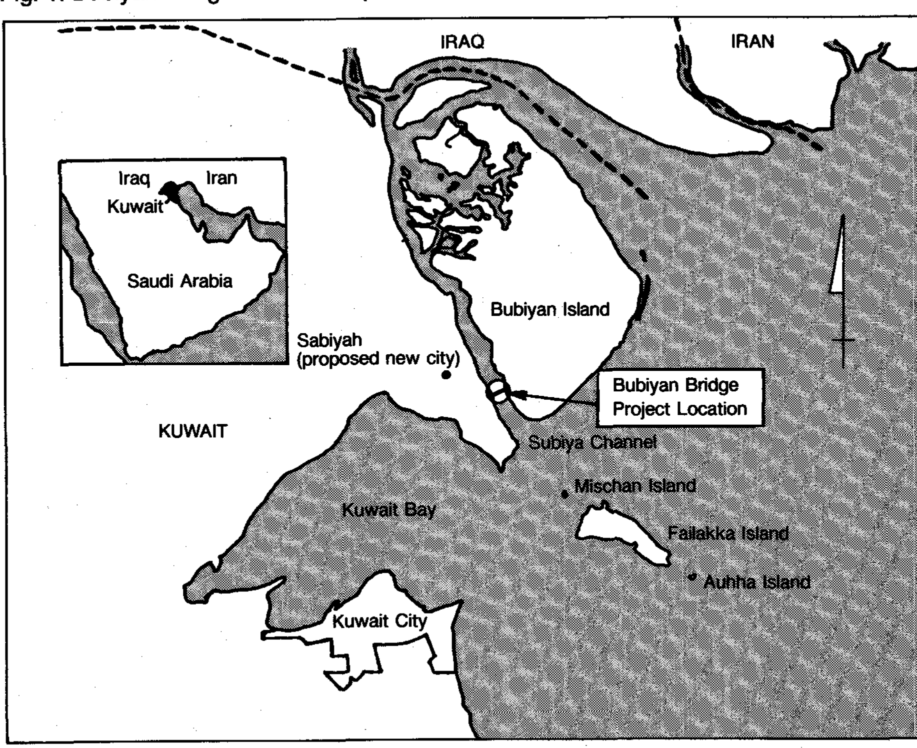
The concept of constructing a highway crossing over the Subiya Channel to connect the mainland of Kuwait to the Island of Bubiyan had been under consideration for a number of years to enhance the potential for development on the island. In September 1979 the Ministry of Public Works (MPW), as a further development of the concept of a bridge to Bubiyan Island, issued a contract to carry out a more definitive planning study.

The resulting "Final Report, Bubiyan Bridge Project, Phase I — Planning — December 1979," included



Bubiyan Bridge during construction.

Fig. 1. Bubiyan Bridge location map.



pertinent data, analysis and recommendations for consideration by MPW, regarding project parameters, design controls and criteria, candidate bridge location alternatives, structural types, and budget estimates.

PROJECT DEVELOPMENT

A review of prequalified firms in the MPW files resulted in the selection of six international firms specializing in the design and construction of major bridges over water. On May 11, 1980, the MPW requested approval from the Central Tenders Committee (CTC) for the concept to negotiate a turnkey project and recommended the six preselected firms. On May 17, 1980, MPW was notified of approval by CTC.

Of the preselected firms, one opted not to participate in the submission of offers. Two other firms chose to participate as a joint venture, resulting in four participating firms. The MPW submitted a "Draft Request for Offers — Bubiyan Bridge Project" to each participating firm for review and comment. All review comments were discussed with the firms at meetings held during the first week of June 1980.

The MPW approved a concept of payment of a stipend to each participating firm, upon submission of an acceptable, responsive offer, to cover cost incurred during the performance of preliminary studies required by the Request for Offers. An official request for offers was issued on June 25, 1980, requiring that offers be submitted to MPW not later than 12 noon, November 25, 1980.

REVIEW AND EVALUATION OF OFFERS

Of the four participating firms, two submitted alternative designs and one provided options to its basic design. A tabulation of basic parameters for each submittal is presented in Table 1, along with a brief description of each concept.

The review of all offers submitted was performed by a panel of engineers of the U.S. Federal Highway Administration (FHWA) who worked in close cooperation with officials of the Ministry of Public Works, Roads and Drainage Department of the State of Kuwait. The FHWA review panel consisted of Joseph DeMarco, Chief, Foreign Projects Division, Washington, D.C., and the authors.

The obvious objective of the review was to determine the best offer as related to responsiveness to the criteria established in the official Request for Offers, relative to other offers submitted. The review panel considered the following five criteria in the ranking and selecting procedure: structural adequacy, future maintenance, aesthetics, time to complete, and cost. All material submitted for each proposal, such as plans, preliminary calculations, specifications, artist's renderings, and models, were carefully reviewed by the panel. In addition, oral interviews were conducted with each firm to answer questions that arose during the review process and to establish that the review panel thoroughly understood the concept of each proposal.

As a result of all reviews and evaluations of the offers submitted, the panel recommended Concept 5 (see Table 1), an externally post-tensioned concrete segmental three-dimensional space frame truss submitted by the French firm of Bouygues. A full report documenting the detailed evaluations and recommendations of the MPW/FHWA review team was submitted in a hand-printed form for expediency and verbally presented at a MPW Major Projects Committee meeting held on December 16, 1980. Subsequently, a formal typed report was submitted by memo to the Chief Engineer, RDD on December 29, 1980, for consideration and concurrence.

Table 1. Tabulation of offers for Bubiyan Bridge project.

Firm	1		2	3			4	
Concept*	1	2	3	4a	4b†	4c	5	6
Total length, m(ft)	2175 (7136)	2175 (7136)	2783 (9131)	2080 (6824)	1925 (6316)	2300 (7546)	2098 (6883)	2098 (6883)
Time to complete, days	1290	1250	1095	517	496	§	700	700
Fixed ceiling cost‡	27.9 (2.54)	28.2 (2.56)	19.0 (1.73)	12.3 (1.11)	11.8 (1.07)	13.2 (1.20)	11.0 (1.00)	12.9 (1.17)

* All concepts included a two lane paved approach road to existing Jahra-Subiya road.
† Required additional causeway embankment of 155 m (509 ft).
‡ Cost given in million Kuwait Dinars (KD), figures in parentheses are relative to successful concept.
§ No time given.

Description of Concepts

1. Precast prestressed concrete segmental box girder main span. Double-T girder approach spans (cast-in-place). Precast concrete piers on cast-in-place concrete piles.
2. Structural steel — six steel girders with composite concrete deck simple and continuous spans. Precast piers on cast-in-place concrete piles.
3. Precast prestressed concrete segmental box girder spans. Each pier consisted of laterally braced extensions of four concrete cast-in-place piles.
4. All three offers include a superstructure of prestressed precast monolith I-girders and deck spans. Piers consisted of precast prestressed concrete cylinder piles with precast caps.
5. Precast concrete segmental 3-D truss space frame, externally post-tensioned. Each pier consists of two hollow cone shapes projecting upward from two solid cylindrical piles cast-in-place below high water elevation.
6. Conventional prestressed precast segmental box girder spans. Pier and foundation are the same as Concept 5 above.

CONTRACT NEGOTIATIONS

Resulting from technical considerations that arose during the proposal reviews, including the various bridge lengths (see Table 1), the MPW requested Bouygues to submit the following additional items of information for consideration during negotiations for a contract:

1. Increase in overall bridge length to achieve:
   (a) Provide at least one span length between abutment and highest water level on both ends of the bridge to allow enough area under the bridge for passage of a maintenance type vehicle behind each abutment.
   (b) Navigation span to be located over the deepest point at the existing natural channel.
   (c) On the Island side, an increase of several span lengths was desirable due to evidence of the tidal area extending over the Island near the shore.

2. A stronger fender system for the navigation channel to allow passage of a Corvette vessel instead of fishing vessels, as in the criteria used for Request for Offers.

3. Items related to an additional requirement of providing for an asphalt wearing course on the bridge.

4. Items related to providing an improved generating plant for the bridge lighting system.

5. Items related to load testing.

6. All other items necessary to respond to other technical questions raised by MPW.

The final contract was for a bridge length of 2500 m (8202 ft) and a navigation clearance of 50 m (164 ft) wide and 20 m (65.6 ft) high above high water at the location of the natural deep channel. Also included is the construction of a two-lane paved approach road approximately 2 km (1.24 miles) long to connect the bridge to the existing Jahra-Subiya road on the mainland, and a short ramp down to existing ground on Bubiyan Island.

The period of construction was estimated to be 775 days. Construction officially started on June 17, 1981.

COST OF PROJECT

The contract price was for a total project firm ceiling of KD 14,096,722 (in Kuwait Dinars). Note that a factor of 1.28 can be used relative to the values presented in Table 1.

The unit cost of the Bubiyan Bridge amounts to about $90.00 (US) per sq ft excluding fenders and approach works. This cost is for construction in Kuwait. Relative to construction prices in the United States, the cost can be estimated to be approximately two-thirds of this value.

BRIDGE CONCEPT

The successful proposal submitted by Bouygues consisted of a three-dimensional truss or space frame concept (Figs. 2 through 4). The concept of prestressed concrete trusses is not new. Concrete trusses have been used in building construction[1,2] and in bridges throughout the world. For example:

1. The Mangfall Bridge in Germany[3,4] is a three-span, cast-in-place, prestressed concrete structure. It may be described simply as a box girder consisting of solid top and bottom flanges connected by two vertical webs, which are trusses.

2. The Rip Bridge in Australia,[4,5,6] just north of Sydney, is a three-span cantilever arch-truss structure. The upper chord (roadway slab), diagonal and vertical truss members, and lower chord are composed of precast elements, which are made integral by cast-in-place concrete and post-tensioning.

3. Other prestressed concrete truss bridges have been constructed in France, the Soviet Union and Japan.

All the structures mentioned above have one aspect in common: the prestressed concrete trusses are all oriented in a vertical plane. The concept is the same as in conventional truss bridges constructed of structural steel members.

The three-dimensional truss concept presented for the Bubiyan Bridge is essentially a multitriangular-cell box girder wherein the longitudinal solid webs are replaced by an open lattice system of trusses. Because the lattice truss webs are oriented in an inclined plane, as opposed to a vertical plane, adjacent trusses have common node points (intersection of diagonal and vertical truss members with the flanges).

This spatial geometry then forms in the transverse direction another system of trusses. Thus, the flanges are connected by a system of inclined orthogonal trusses (a system of mutually perpendicular trusses) as shown in Fig. 5. Because the trusses are inclined to each other, with the diagonal and vertical members intersecting at common node points, they form a space frame composed of interconnecting pyramids and half-pyramids. Thus, the structural behavior of the bridge with regard to distribution of load resembles that of a two-way slab in building construction.

This structural concept is new in its application to a bridge structure. However, the concept of a space frame truss has been previously applied to roof

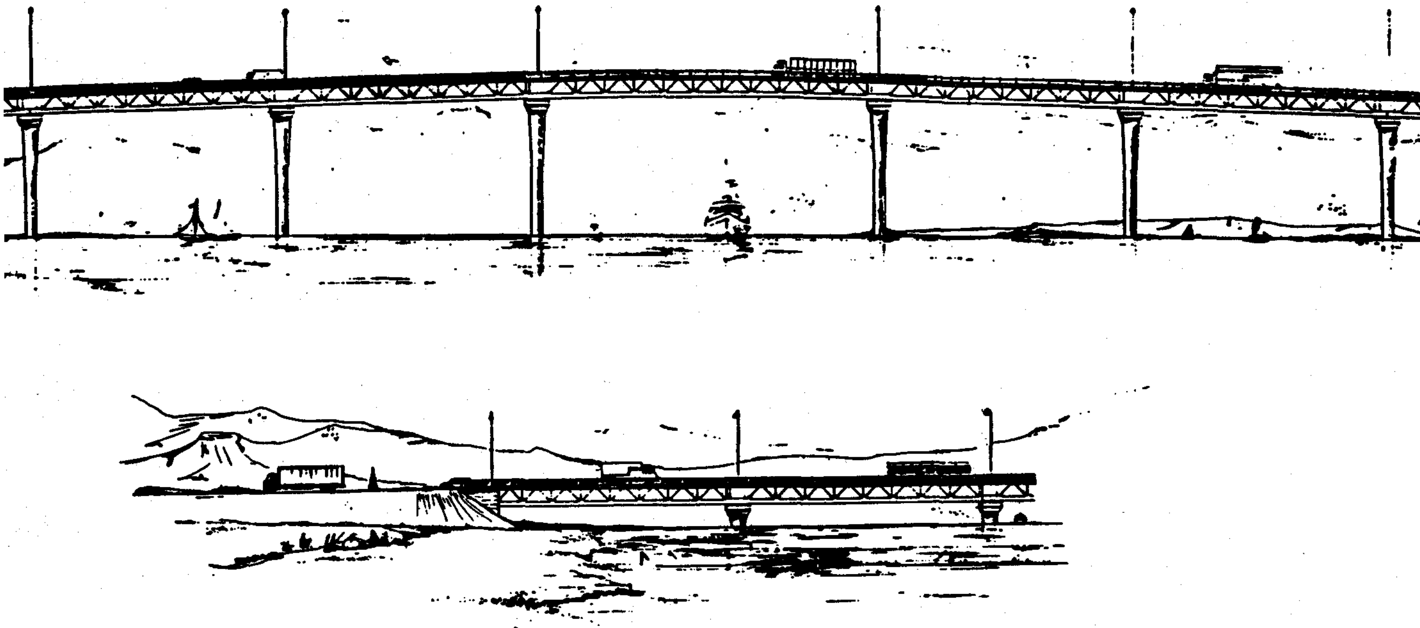


Fig. 2. Elevations of Bubiyan Bridge (prepared by Bouygues).

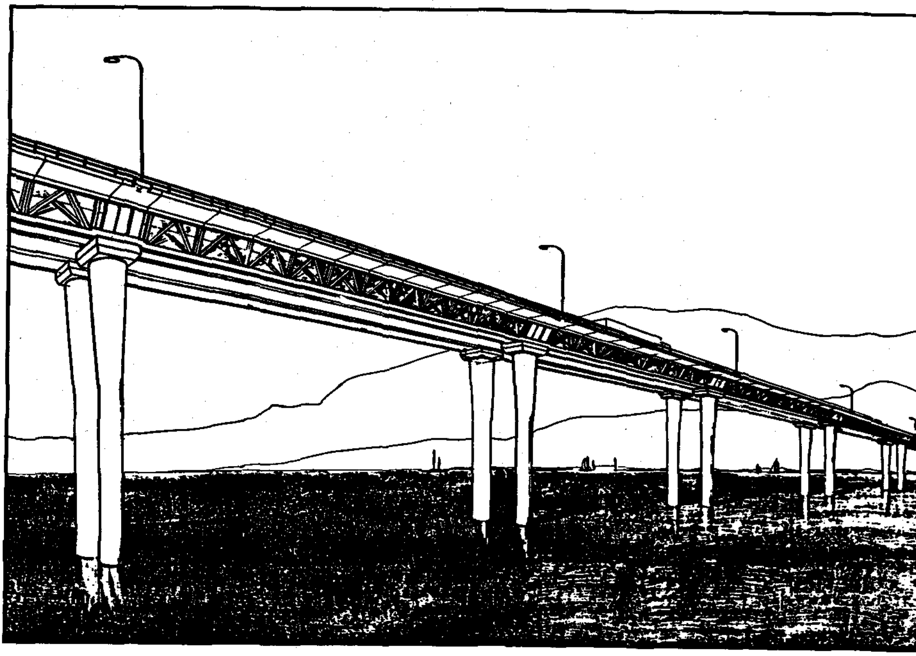


Fig. 3. Artist's sketch of Bubiyan Bridge (prepared by Bouygues).

structures for large column-free sport facilities, auditoriums, civic centers, and the like. These space structures have been constructed primarily of metallic (steel or aluminum) tubular sections. There was no reason to believe that, with the current state-of-the-art in prestressed concrete, segmental construction, and existing concrete truss construction, a prestressed concrete space frame concept could not be consummated — in particular for a bridge structure.

The fabrication and erection of the superstructure is consistent with state-of-the-art conventional prestressed precast segmental construction, including the external prestressing. Although the concept of a space frame structure is new to bridge construction, its uniqueness is only in assembling existing concepts into a single concept.

Figs. 3, 4, and 5 show an artist's sketch, construction model, and typical segment of the bridge.

PROJECT DESCRIPTION

Total length of structure, from centerline of Mainland abutment bearing to centerline of Bubiyan Island abutment bearing, is 2503.05 m (8212.11 ft). This length of structure is divided by expansion joints into 12 units of either five or six continuous spans (see Fig. 6 and Table 2). Thus, there are a total of 62 spans. Transversely, the superstructure has a width of 18.2 m (59.7 ft) which accommodates two roadways of 4.3 m (14.1 ft) with shoulders of 2.3 m (7.5 ft) and two sidewalks of 1.5 m (4.92 ft), as shown in Fig. 7(a).

Horizontal and vertical alignments are in accordance with AASHTO: "A Policy on Geometric Design of Rural Highways, 1965." Vertical curvature has a radius of 10,000 m (32,800 ft) in crests and 5000 m (16,400 ft) in sags. Maximum grade is 3 percent and design speed is 120 km/hr (75 miles per hour).

Each pier consists of twin 1.792 m

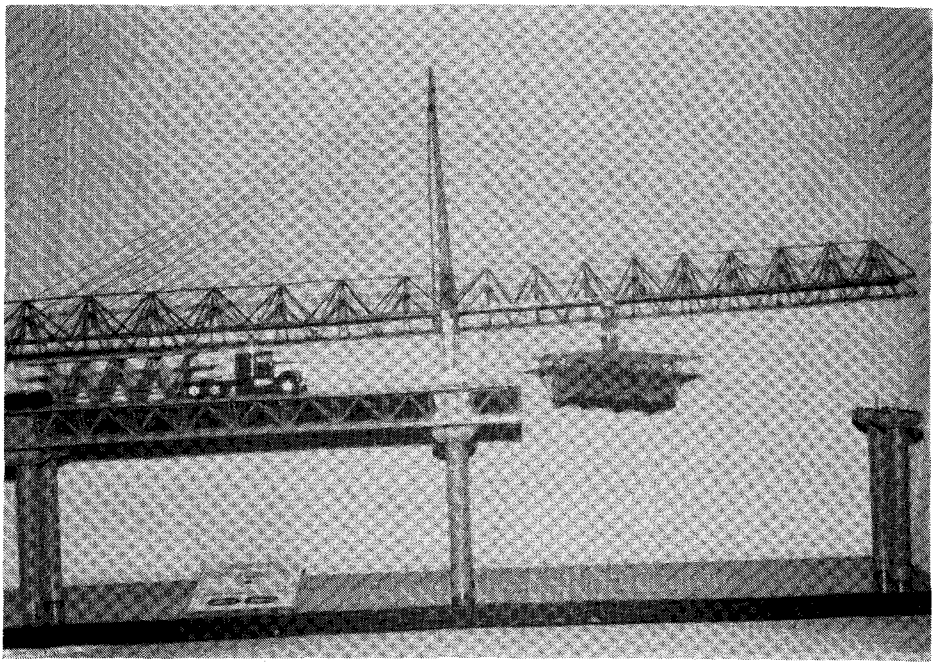


Fig. 4. Construction stage model of Bubiyan Bridge.

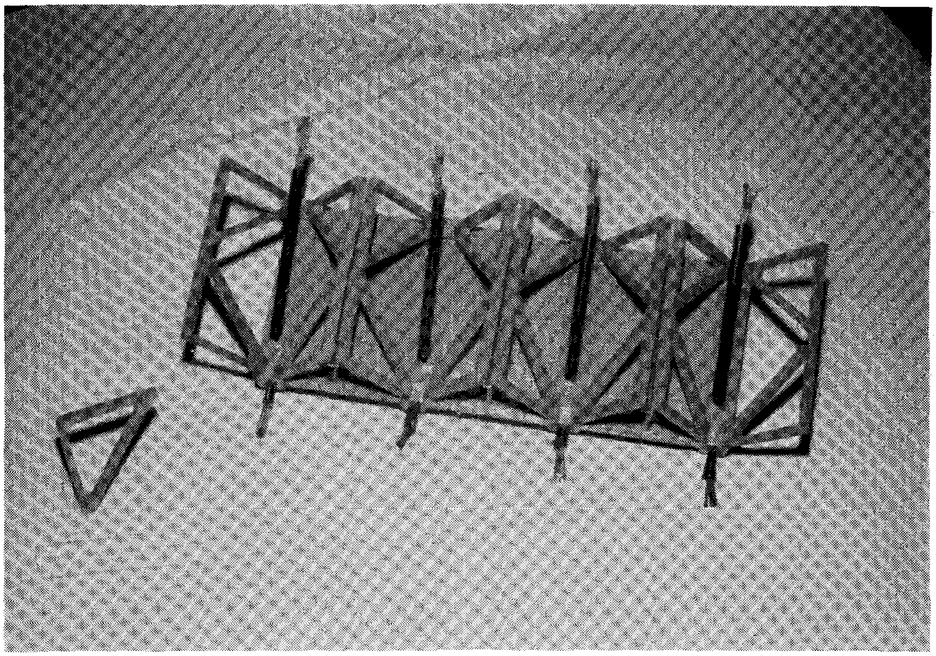


Fig. 5. Model of typical segment with top flange removed (Bubiyan Bridge). Note external post-tensioning tendons.

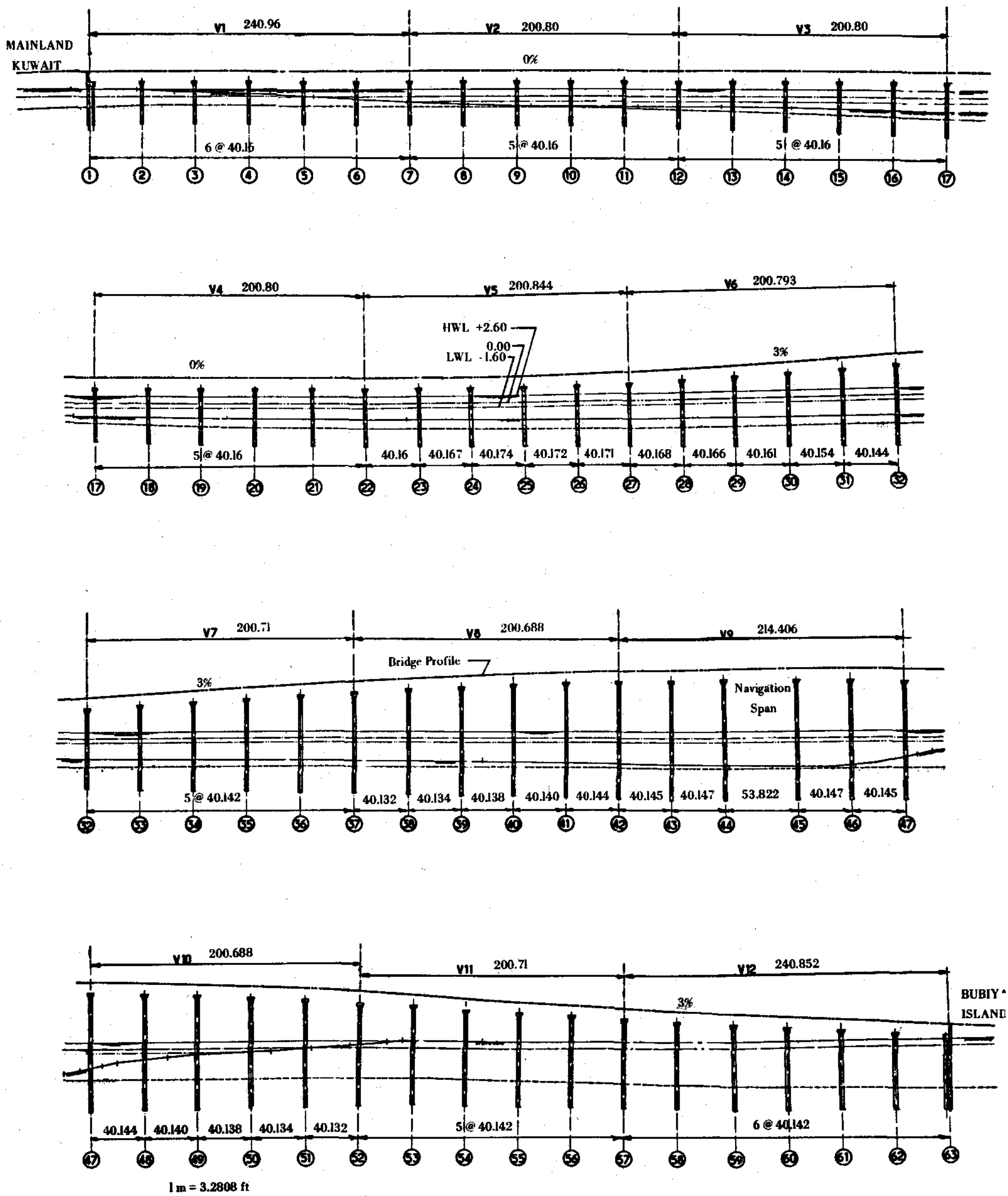
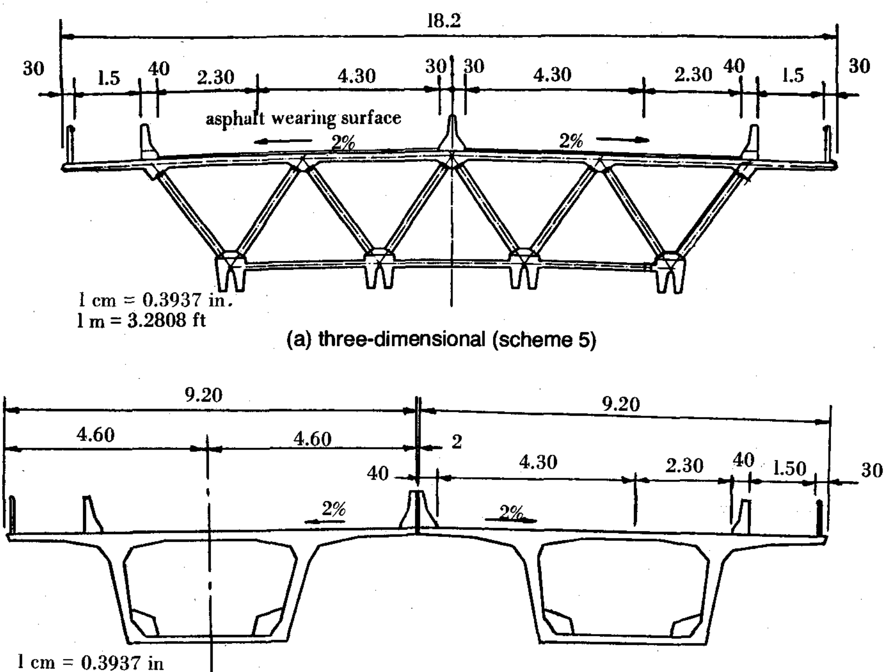


Fig. 6. Elevation of Bubiyan Bridge showing span arrangement. See Fig. 34 for final design modifications to structure for Bubiyan Island ramp.

Table 2. Tabulation of spans, Bubiyan Bridge, m (ft).

Unit	Span dimensions in unit	Unit length	
V1	6 @ 40.16 (131.7585)	240.96	(790.5511)
V2	5 @ 40.16 (131.7585)	200.80	(685.7927)
V3	5 @ 40.16 (131.7585)	200.80	(658.7927)
V4	5 @ 40.16 (131.7585)	200.80	(658.7927)
V5	40.16 – 40.167 – 40.174 – 40.172 – 40.171 (131.7585 – 131.7815 – 131.8045 – 131.7979 – 131.7946)	200.844	(658.9370)
V6	40.168 – 40.166 – 40.161 – 40.154 – 40.144 (131.7848 – 131.7782 – 131.7618 – 131.7388 – 131.7060)	200.793	(658.7697)
V7	5 @ 40.142 (131.6995)	200.71	(658.4974)
V8	40.132 – 40.134 – 40.138 – 40.140 – 40.144 (131.6667 – 131.6732 – 131.6864 – 131.6929 – 131.7060)	200.688	(658.4252)
V9	40.145 – 40.147 – 53.822 – 40.147 – 40.145 (131.7093 – 131.7159 – 176.5814 – 131.7159 – 131.7093)	214.406	(703.4318)
V10	40.144 – 40.140 – 40.138 – 40.134 – 40.132 (131.7060 – 131.6929 – 131.6864 – 131.6732 – 131.6667)	200.688	(658.4252)
V11	5 @ 40.142 (131.6995)	200.71	(658.4974)
V12	6 @ 40.142 (131.6995)	240.852	(790.1969)
	Total length:	2503.05	(8212.1096)

Note: See Table 4 for final design modifications to structure for Bubiyan Island ramp.



(a) three-dimensional (scheme 5)

(b) box-girder (scheme 6)

Fig. 7. Alternate superstructure cross sections.

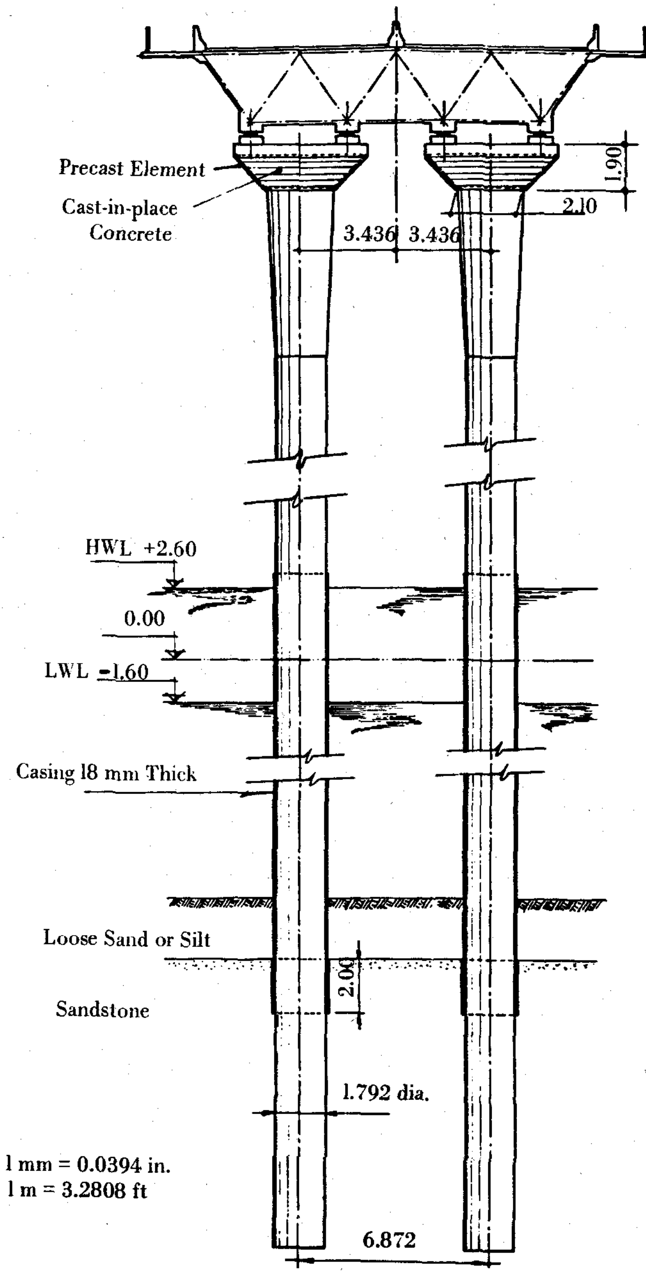


Fig. 8. Typical pier arrangement.

Fig. 9. Typical twin pier shafts (courtesy of Bouygues).

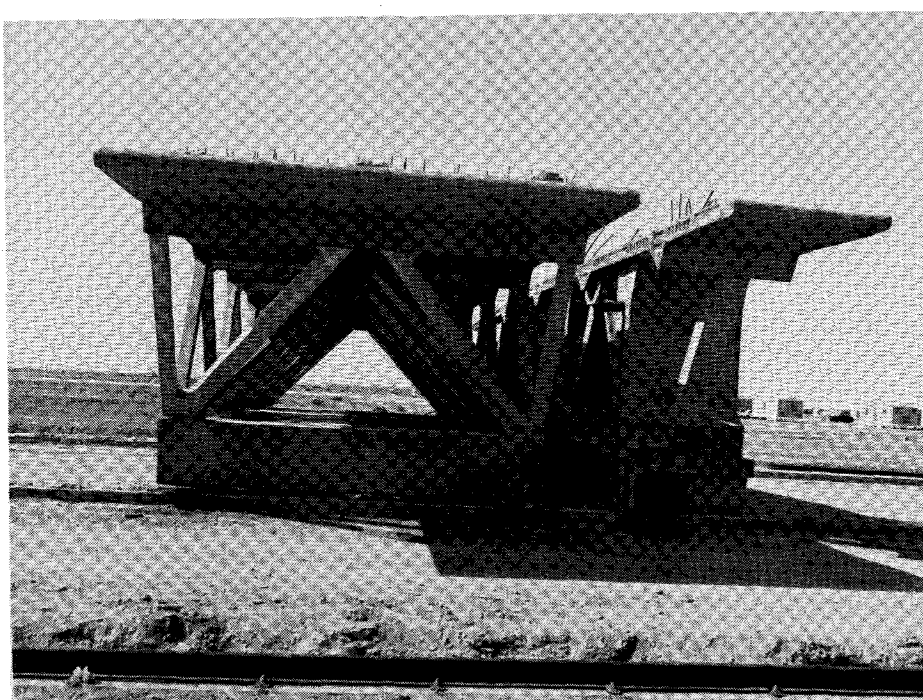


Fig. 10. Typical segment and pier segment (courtesy of Bouygues).

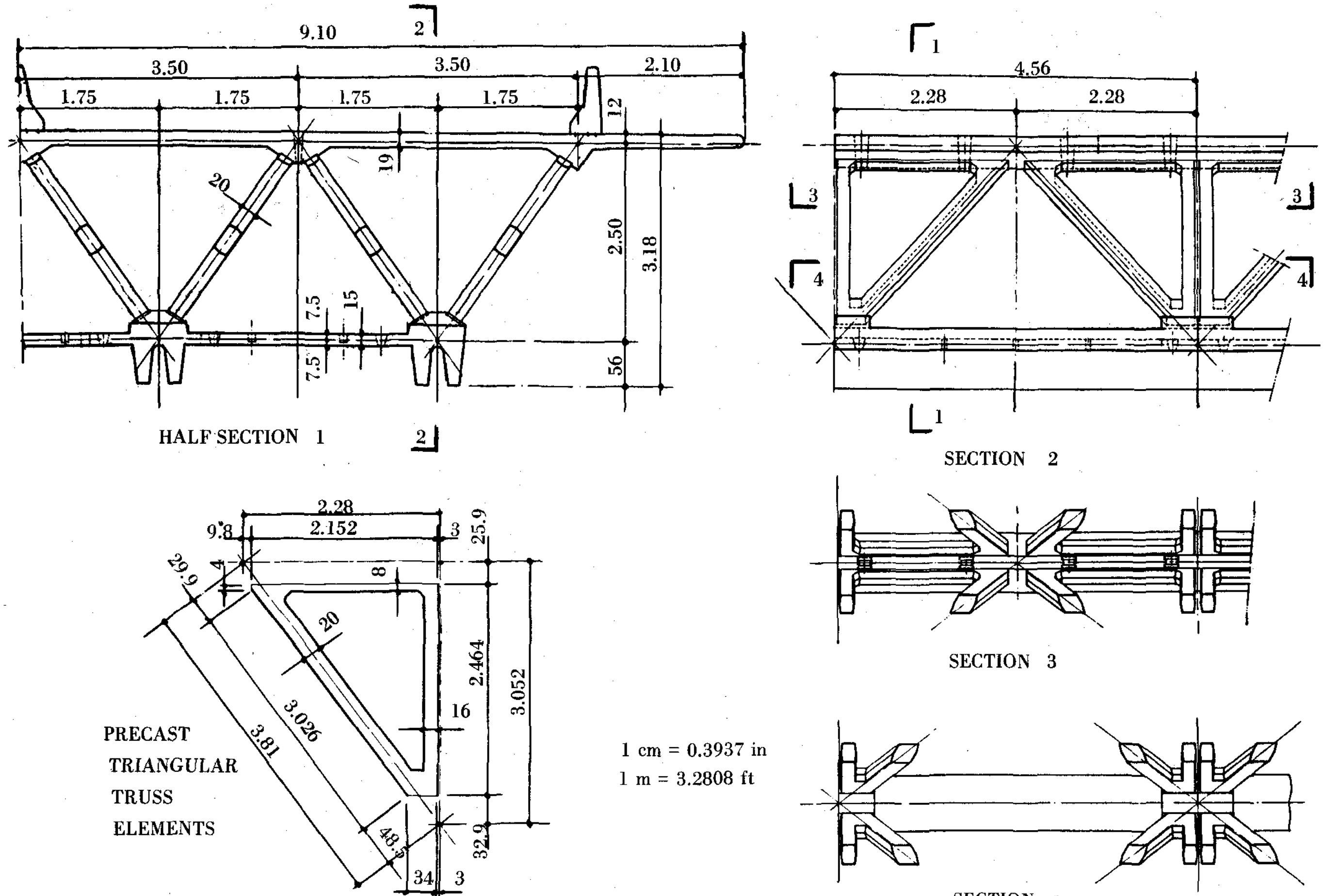


Fig. 11. Details of a typical segment showing various sections.

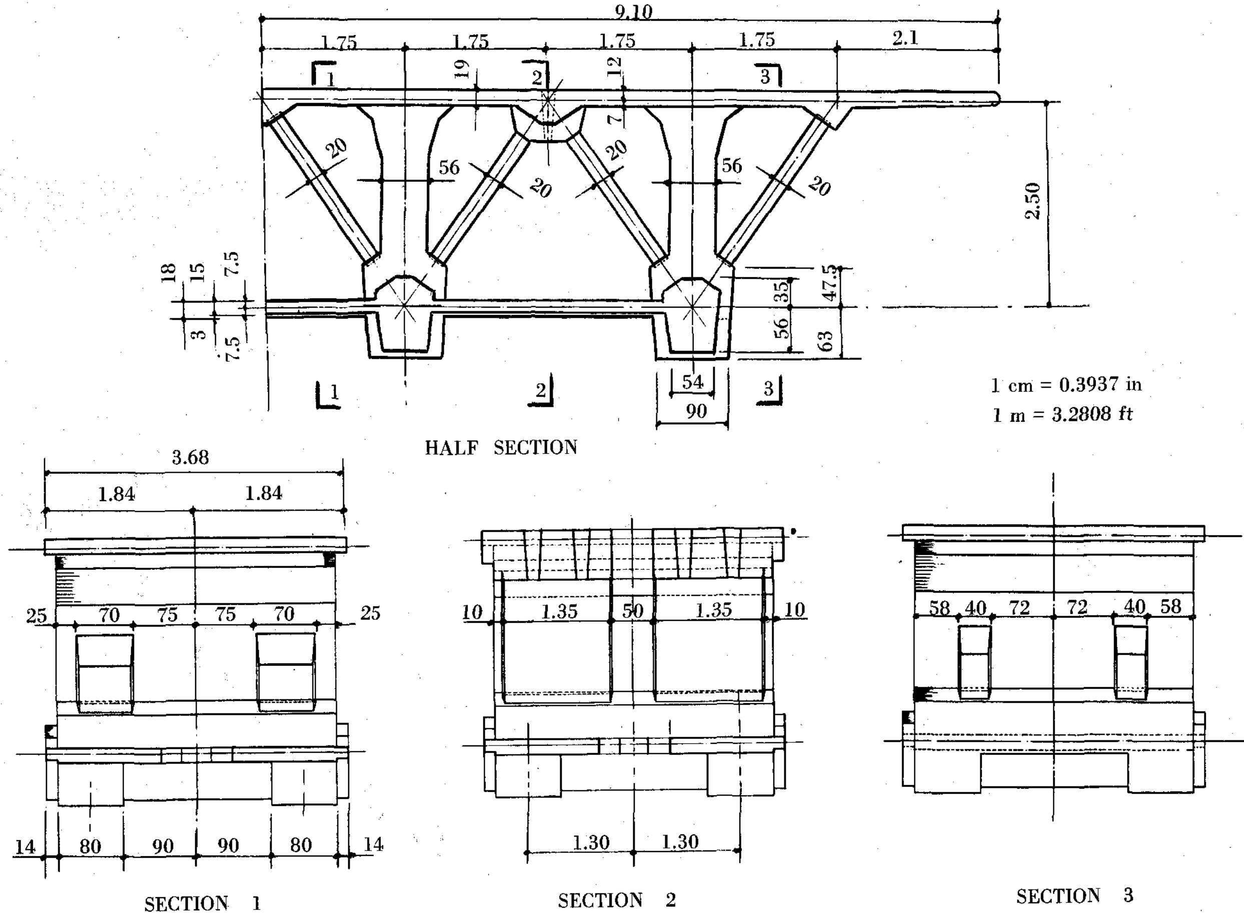


Fig. 12. Details of a pier segment showing various sections.

(5.88 ft) diameter shafts spaced at 6.872 m (22.55 ft) (see Figs. 8 and 9). Each shaft is constructed by driving a 18 mm (0.7 in.) "weathering" steel casing 2 m (6.6 ft) into sandstone, excavating below the casing level to the proper elevation to provide the socket required, placing reinforcement, and concreting to the underside of the pier caps. Conventional forming techniques are used from the top of the steel casing to the underside of the pier caps. The steel casing is considered as a stay-in-place form and is not considered to act structurally (other than for erection loads).

Piers for low level units (V1 through V5) are socketed 7 m (23 ft) into the sandstone. Maximum socket length in the highest unit (V9) is 12 m (39.37 ft). Socket length for other piers varies linearly from 7 to 12 m (23 to 39.37 ft) as a function of the pier height. Pier caps consist of a precast stay-in-place form shell that is then filled with reinforced concrete.

Each span of the structure, with the exception of the navigation span, consists of eight typical segments and the pier segments (see Fig. 10). The navigation span contains eleven typical segments. Details of a typical segment are presented in Fig. 11. A typical segment basically contains a top flange of 19 cm (7.48 in.) thickness, a bottom flange of 15 cm (5.9 in.) thickness, and precast triangular elements that form the space frame truss members connecting the two flanges. The pier segment (Fig. 12) contains heavy vertical web elements (shaped similar to an I-girder) which accommodate the post-tensioning anchorages, and diagonal slab elements whose size is a function of stress requirements.

Prestressing is accomplished by external post-tensioning in a manner similar to that utilized in the Long Key[4,7,8] and the Seven Mile Bridges in the Florida Keys. Typical spans are post-tensioned by eight tendons and the navigation span by twelve tendons. Each tendon consists of 24 strands. Each strand has an area of 139 mm² and an ultimate strength of 1814 MPa (0.6 in. diameter 263 ksi strand). Tendons are continuous in a five or six span unit. Tendons in an individual span are made continuous with the previous span by a coupling located forward (in the direction of erection) of each pier segment unit (Fig. 13). All tendons are sheathed in polypropylene ducts which are reinforced with an internal metallic liner at each harp point.

The tendon profile is similar to that of pretensioned girders with two harping points (Fig. 13). There are two tendons per longitudinal bottom flange rib, except for the longer navigation span where there are three tendons per rib. It can be seen from Fig. 13 that in a typical span the upper tendon for all four ribs is harped at Joint 4. The bottom tendon of the exterior ribs is harped at Joint 2 and the bottom tendon for the interior ribs is harped at Joint 3. The post-tensioning profile is selected such that there will be no stress reversals in the diagonals under service load.

COMPARISION OF 3-D CONCEPT WITH CONVENTIONAL BOX GIRDER

In response to the Request for Offers, Bouygues prepared a study (drawings and calculations) of an alternative solution (Concept No. 6, Table 1). This alternate differs from the basic 3-D space frame alternate only with respect to the cross section of the superstructure, which consists of two single-cell conventional segmental box girders [see Fig. 7(b)]. Top and bottom flange thicknesses are 0.20 and 0.18 m (8 and 7 in.), respectively, and the average web thickness is 0.28 m (11 in.).

External prestressing tendons are located inside the box cells and are an-

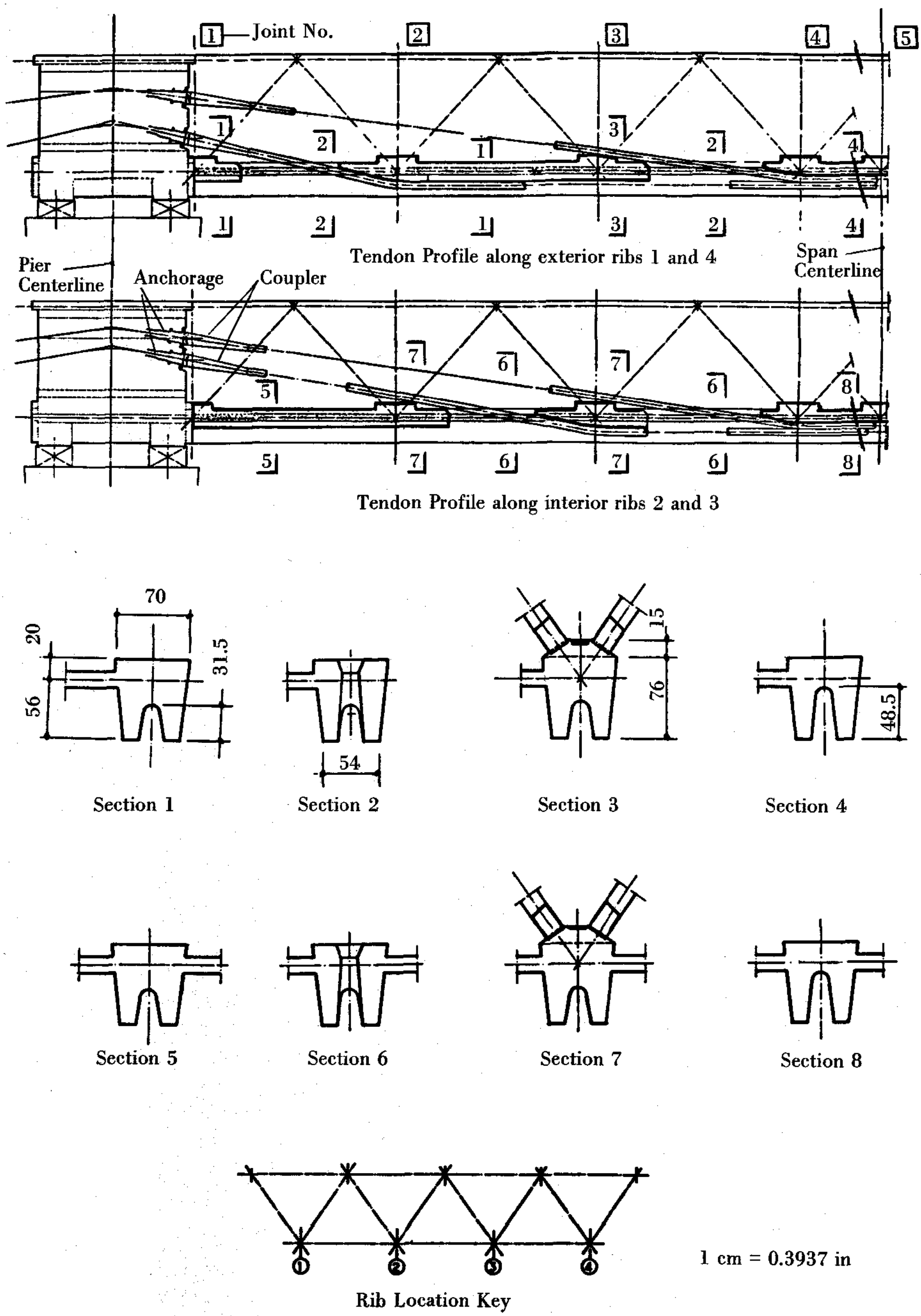


Fig. 13. Tendon profile in a typical span showing various section details.

Table 3. Comparison of substructure.

Reactions, moments, displacements	Space frame	Box girders	Percent change*
Max. reaction at top of pier, T (kips)	780 (1720)	930 (2050)	+19.2
Max. moment at top of high piers, T-m (k-ft)	730 (5280)	903 (6529)	+23.7
Deck displacement from seismic load, cm (in.)	11 (4.33)	14 (5.51)	+27.3
Max. reaction in sandstone, T (kips)	930 (2050)	1080 (2381)	+16.1
Max. moment in piles at sandstone level, T-m (k-ft)	390 (2821)	480 (3472)	+23.1

* With space frame as base.

chored at the pier diaphragms. Change in tendon profile is accomplished by deviation saddles located at the intersection of the webs with the bottom flange; the basic concept is the same as that in the Long Key and Seven Mile Bridges.

The box girder alternate is equivalent to the space frame alternative with respect to overall length, span dimensions, width, depth, and navigation clearances. Therefore, an opportunity presents itself for comparison of the two alternates from an economical and technical viewpoint.

As previously indicated in Table 1, the box girder alternate was 17 percent more expensive than the space frame alternate. The dead weight of the superstructure (exclusive of wearing surface, barriers, etc.) is 18.49 T/m (12.42 kips per ft) for the space frame and 21.32 T/m (14.33 kips per ft) for the box girders, an approximate 15 percent increase over the space frame.

Geometric characteristics of the piers are identical for both alternatives, taking into account the bearing capacity of the sandstone foundation material. The embedment length of the piles in the sandstone and reinforcement steel in the piers and piles are controlled by seismic loads. Table 3 presents a comparison tabulation of various substructure parameters. Since the box girders have an increased mass as compared to the space frame, it is penalized under seismic loading conditions.

From the data presented above, the advantage of the space frame concept is obvious. Further, there is an advantage in the manner the load is distributed throughout the superstructure. That is to say, there are many load paths. In the unforeseen event of a member failure, the load would redistribute itself by seeking an alternative load path. Therefore, there is a greater degree of redundancy.

SUPERSTRUCTURE DESIGN

The design of the Bubiyan Bridge was based on the AASHTO Standard Specifications for Highway Bridges and the Deutsche Industrie Normen (DIN) design loads — DIN 1072, Bridge Class (Bruckenlasse) 60. AASHTO design was for the service load design method with HS20-44 live load. Because of the higher vehicle loads in Kuwait, the stresses determined by AASHTO were then analyzed for compliance with the relevant DIN Sections. The specification requiring the larger member size controlled.* The design was then checked for ultimate strength for special Kuwait trailer loadings (tanks). AASHTO Specifications were used for this analysis.

Structural design also included con-

*Comparison between the two specifications became somewhat cumbersome and it has been suggested to the MPW that in the future the AASHTO Specifications be used exclusively with an increase of live load to HS-30 to accommodate heavier vehicle loads.

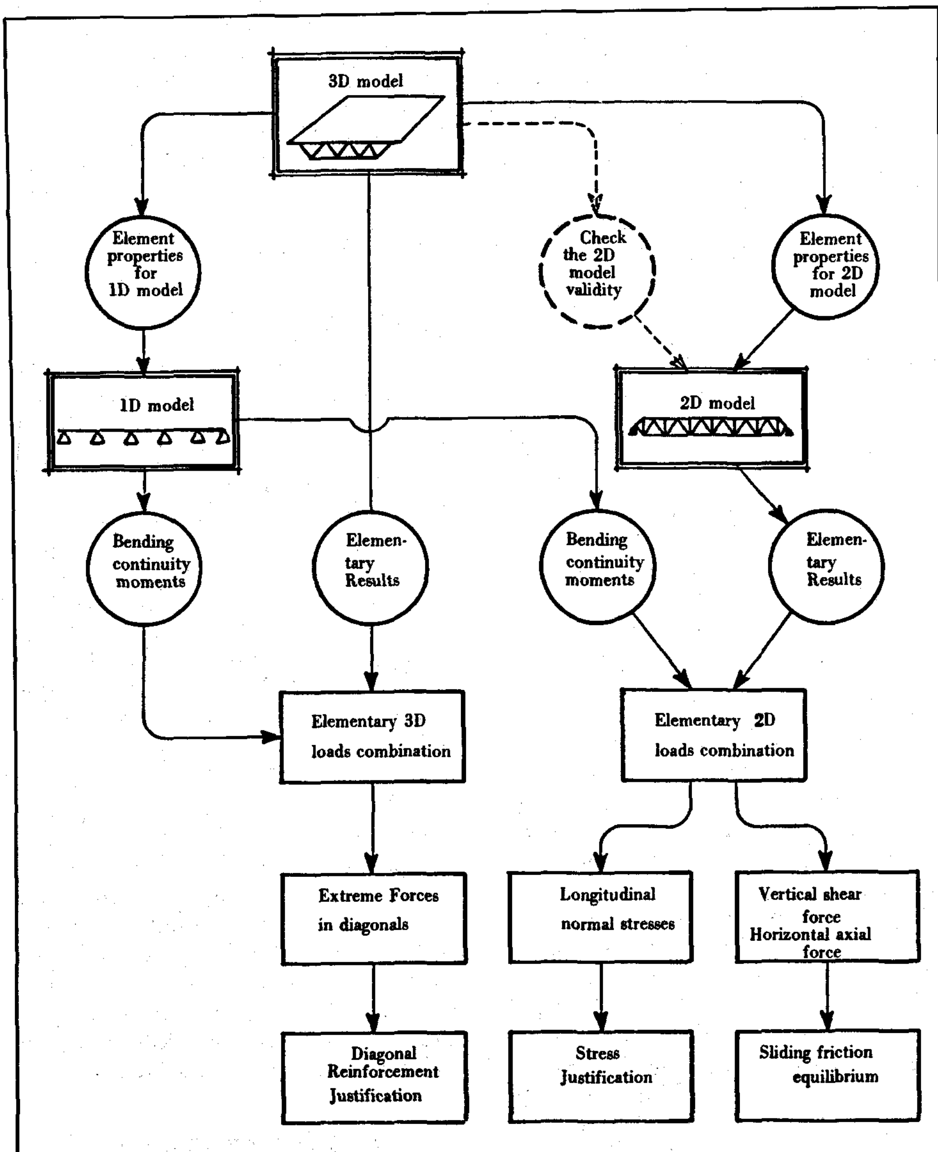


Fig. 14. Linking of FASTRUDL computer models.

sideration of a future utilities loading of 500 kg/m (336 lb per ft).

Seismic investigations indicated that a Uniform Building Code (UBC) Zone 1 classification, equivalent to a maximum horizontal acceleration of 0.10g, was appropriate for the site. Taking into account the structure period and depth to rock (AASHTO Section 1.2.20), a response coefficient of 0.06 was assumed.

Design for creep, shrinkage and temperature effects were based on the French Prestress Code IP2 which is in conformity with International Recommendations (published by CEB-FIP in 1978).

No epoxy was specified in the joints between segments; that is, the design is based on dry joints. This means that there must be permanent compression along the plane of the joint, i.e., no tension. These joint planes are checked to insure that they are non-sliding.

This requirement is expressed by the criterion, $T/N \leq 0.3$, where T denotes the shear force in the section and N the normal force. The coefficient of friction of concrete on concrete is assumed to be a minimum of 0.6 which provides a factor of safety in excess of two for sliding in the above equation.

Compression diagonals are designed on the basis of an assumption of an initial eccentricity of forces of 2 cm (¾ in.). Tension diagonals are designed such that all tension is taken by the reinforcing steel.

Analysis of the space frame superstructure was conducted using the FASTRUDL Structural Computer Code (derived from the MIT STRUDL Code) as developed in France by the French Metal Structures Organization (CTICM) for offshore structures. For the general flexural analysis three FASTRUDL models were used; a 1D, 2D and 3D version (see Fig. 14).

In the 3D model, a typical span is modeled in space with all the diagonals and verticals of the typical segments represented as bar elements. Slabs, as well as the vertical and diagonal diaphragms in the pier segments, are represented by four node plate elements. All nodes of the model have six DOF (degrees of freedom) nodes. Boundary conditions are defined such that four nodes at each of the span extremities are restrained in the vertical direction.

In the 2D model, one typical span is modeled in the vertical axis plane with the nodes at the span limits restrained in the vertical direction (simple span condition). In the 1D model, six (or five) spans (between expansion joints) are modeled as a continuous beam which includes shear deflections.

The linking of the three models is indicated in the flow chart of Fig. 14. The 1D model is used to compute all continuity bending moments which are applied to the 2D and 3D models. Inertia and shear area for the 1D model are defined from the 3D model such that rotations and vertical deflections under unit loads are the same in the two models, assuring that the 1D model is representative of the actual structure.

The 3D model allows full structural analysis under any loading condition, particularly transverse bending. From the various transverse loading positions, the extreme diagonal forces are determined. For each span of a five (or six) span unit the continuity bending moments determined by the 1D model are applied to the ends of the 3D model. The combined effect provides the extreme force effects for each diagonal.

Properties for the 2D model are taken such that rotations and deflections under unit loads are the same as the 3D model. Loads from the 3D model and the bending continuity moments from the 1D model are applied to the 2D model to determine longitudinal normal forces and bending moments in the top and bottom slab. In addition, vertical and axial forces are determined at each joint to check the sliding friction.

FABRICATION OF SUPERSTRUCTURE SEGMENTS

Fabrication of the precast superstructure basically requires four operations:

(1) Production of the precast triangular truss elements for the typical segments (Figure 11); (2) the slab diagonals for the pier segments (Figure 12); (3) the pier segments; and lastly (4) the typical segments.

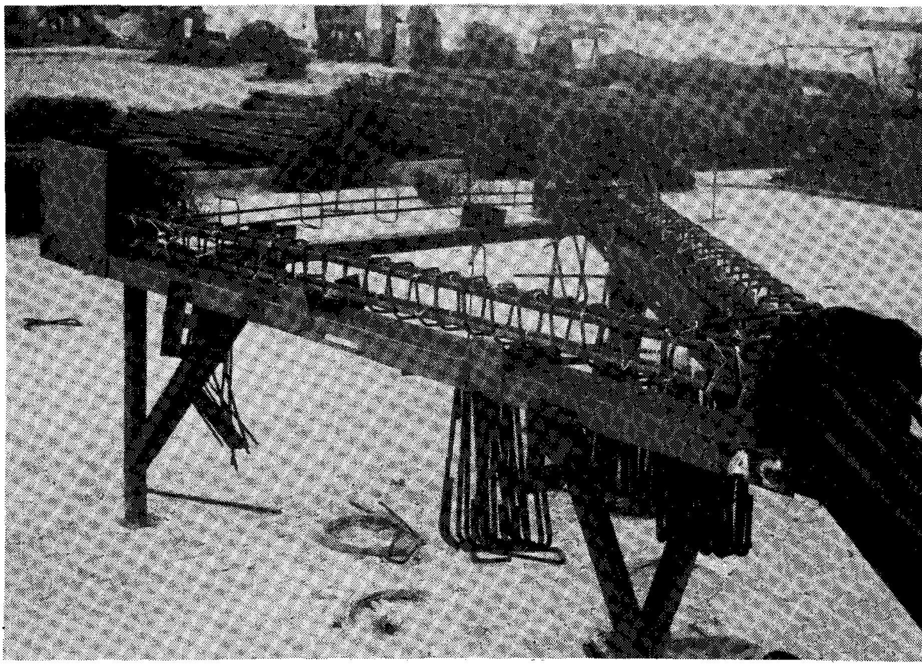


Fig. 15. Fabrication of triangle reinforcement cage.

Precast Triangles

Reinforcement is prefabricated into cages (see Fig. 15) which are transported to the casting area and stored vertically on racks. The triangles are cast in a vertical position in metal forms (Fig. 16), which are supported on three supports. Twenty-six triangles are cast simultaneously with concrete being delivered by a tower crane.

A plasticizing additive is utilized in the concrete to insure proper workability and filling of the form. In addition, external form vibrators as well as internal needle type vibrators are used to enhance the concrete placement. After concreting, the tops of the forms are covered with a few centimeters of water to avoid drying out. The triangles are stripped after 16 hours and stored vertically on framework racks (see Fig. 17).

During the first 2 days of storage the triangles are protected from the sun by tarpaulin covers. Triangles are cured for

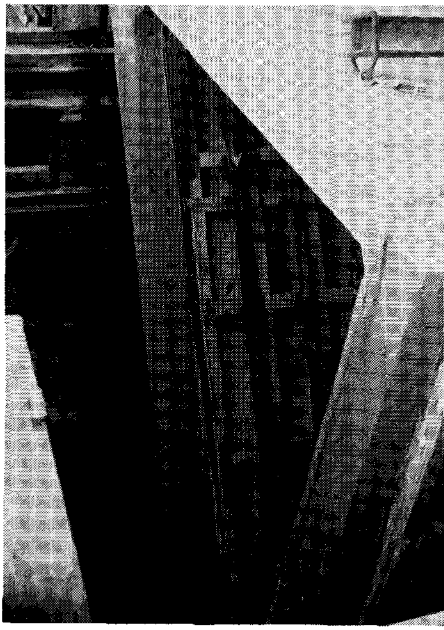


Fig. 16. Triangles are cast vertically in metal forms. They are then stripped after 16 hours and stored vertically.

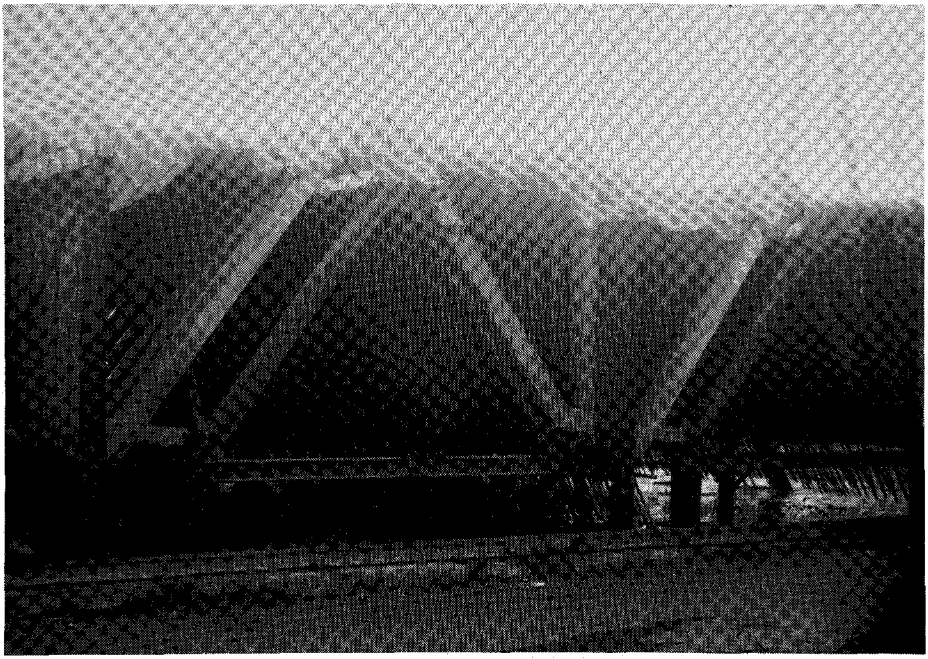


Fig. 17. Precast triangles in storage racks.

a minimum of 1 week before being placed in the typical segment forms. A schematic sequence of operations for triangle production is given in Fig. 18.

Pier Segment Diagonals

Pier segment slab diagonals are prefabricated in essentially the same manner as the triangles described above. The narrow diagonals (Sections 1 and 3 of Fig. 12) are cast in a horizontal position and are stored horizontally on wooden sleepers. The large slab diagonals (Section 2 of Fig. 14) are cast as a set in a vertical position as indicated by the schematic of operations given in Fig. 19. These units are shown in the storage yard in Fig. 20.

Pier Segments

Pier segments are actually fabricated as two match-cast half segments. Thus, the expansion joint segments have the same sequence of fabrication as the other pier segments with the obvious exception of accommodating the expansion joint detail. The forms are such that the bottom form under the vertical webs is one continuous piece to accommodate both half segments. The balance of the formwork accommodates a half segment and is moved from one-half segment to the other following the sequence of casting. Since the pier segment is tilted to follow the slope of the structure and since the bearing areas are level, adjustable boxes, for forming the bearing blocks, are incorporated inside the form to conform to the proper geometry of the bearing surface.

The sequence of casting operations is as follows:

Day 1 1. Stripping of half Segment A and transfer to storage [Fig. 21(a)].

2. Stripping of half Segment B bottom flange forms and

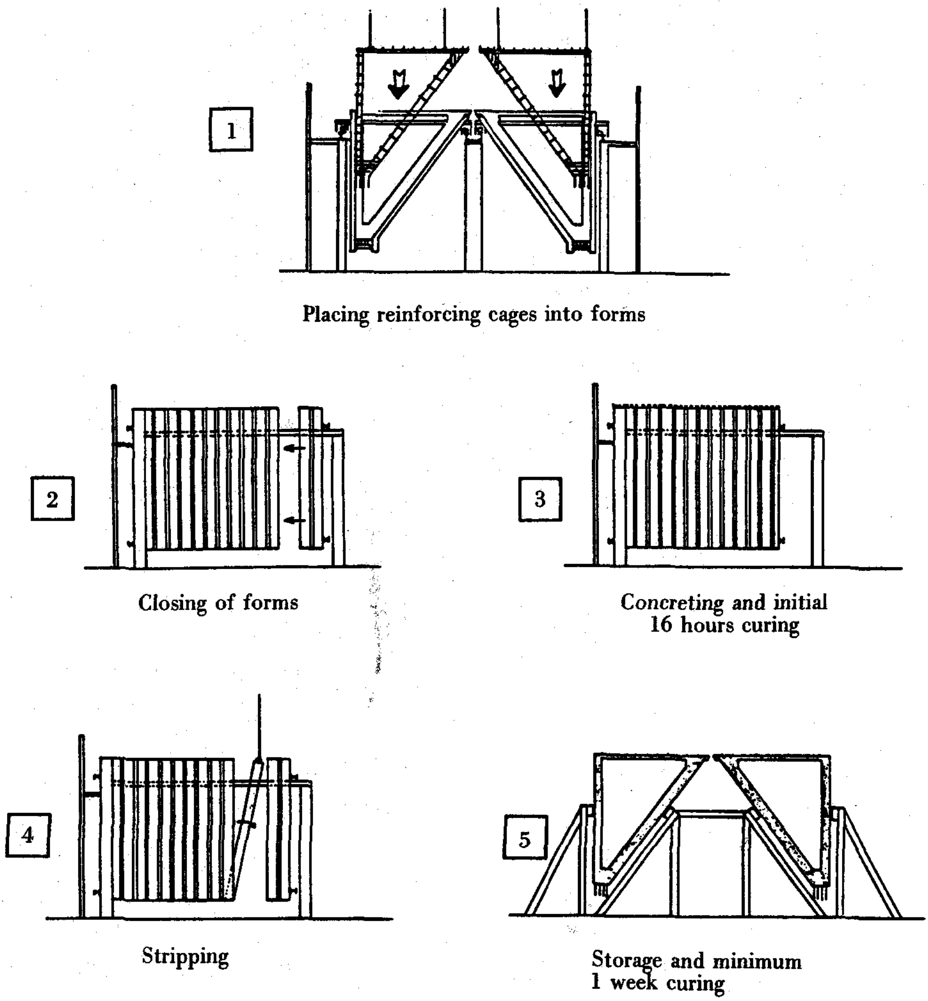


Fig. 18. Schematic of triangle fabrication.

transferring to half Segment A[Fig. 21(b)].

3. Placing reinforcement in webs of half Segment A [Fig. 21(b)].

Day 2
1. Reinforcing of lower deck, half Segment A [(Fig. 21(c)].
2. Installation of precast diagonal elements [Fig. 21(d)].
3. Completion of reinforcement in lower nodes.
4. Transfer of vertical web and top flange forms from half Segment B to half Segment A[Fig. 21(e)].

Day 3
1. Reinforcing of top flange [Fig. 21(f)].
2. Placing bulkhead form.
3. Concreting [Fig. 21(g)].

Day 4
1. Remove half Segment B to storage.
2. Repeat above cycle to produce a new half Segment B.

Pier segment forms are shown in Fig. 22.

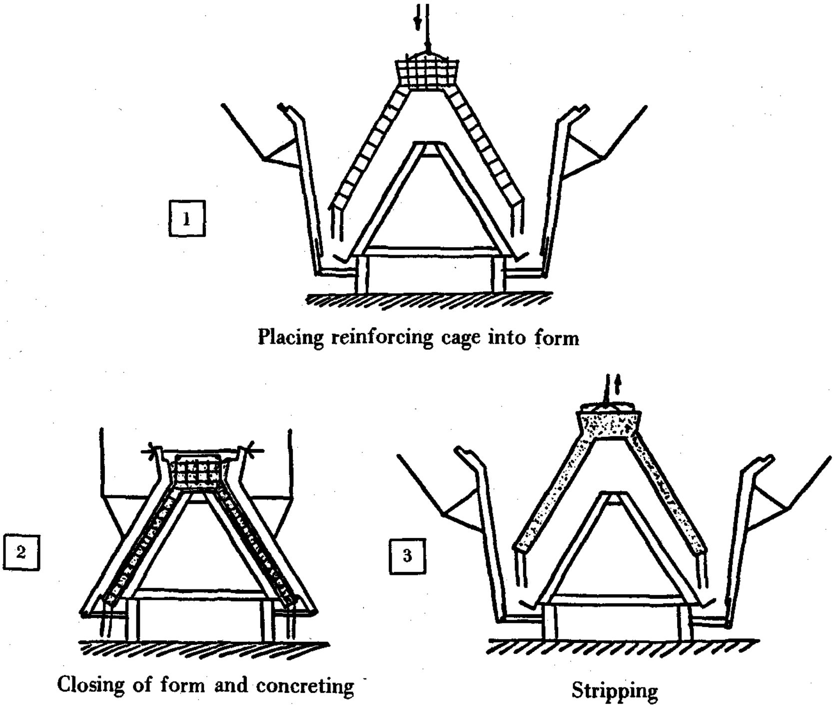


Fig. 19. Fabrication schematic of pier segment diagonals.

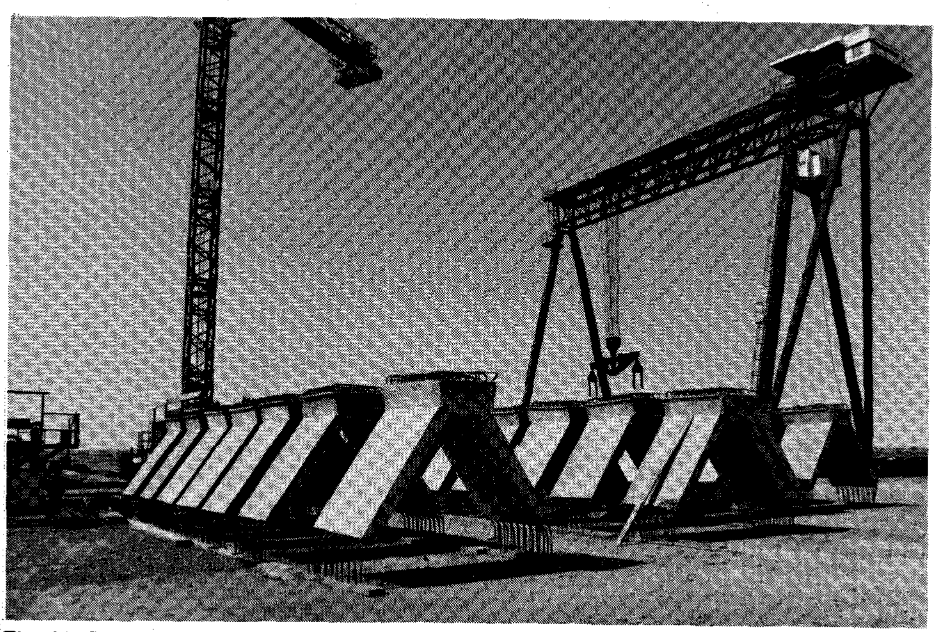


Fig. 20. Pier segment diagonals in storage yard (courtesy of Bouygues).

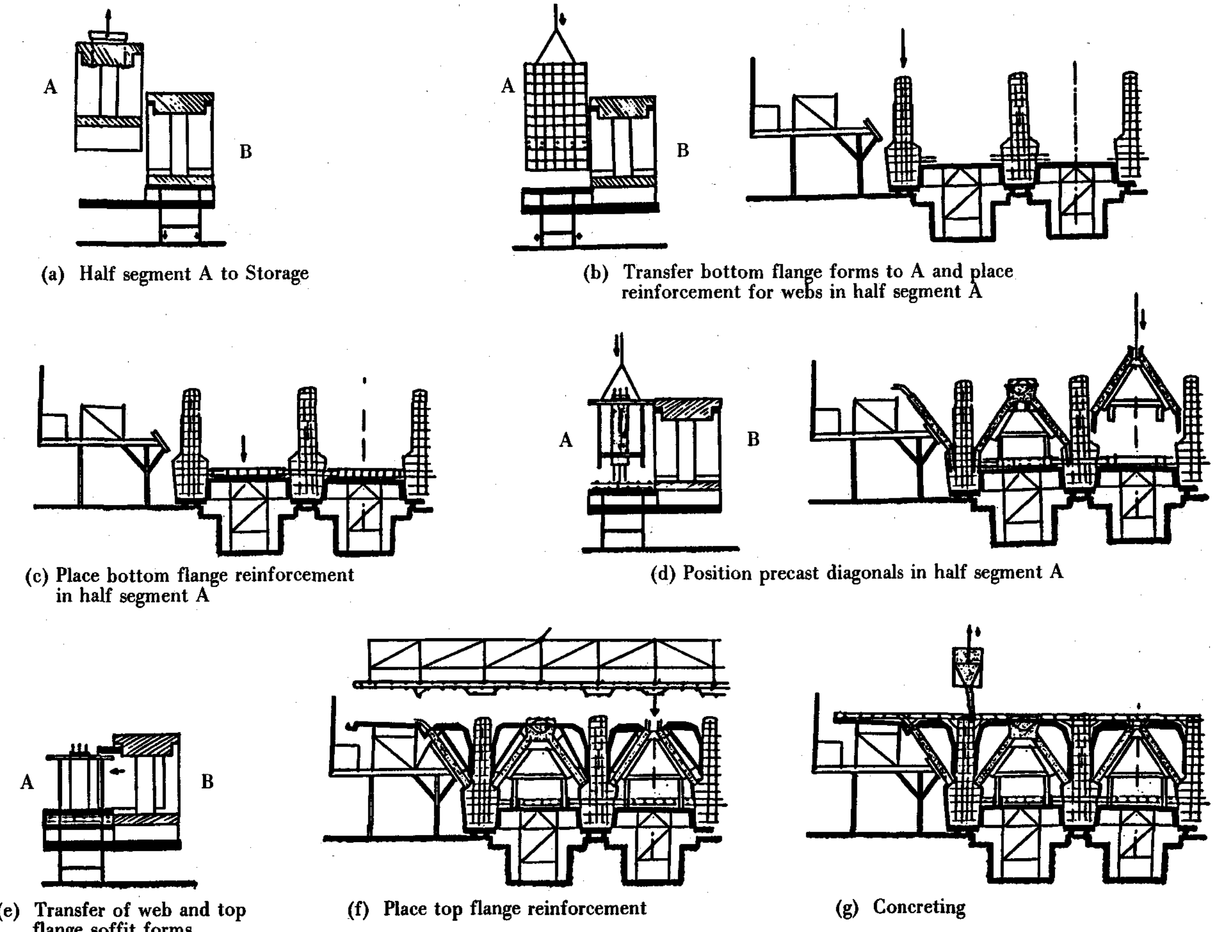


Fig. 21. Schematic of pier segment casting operations.

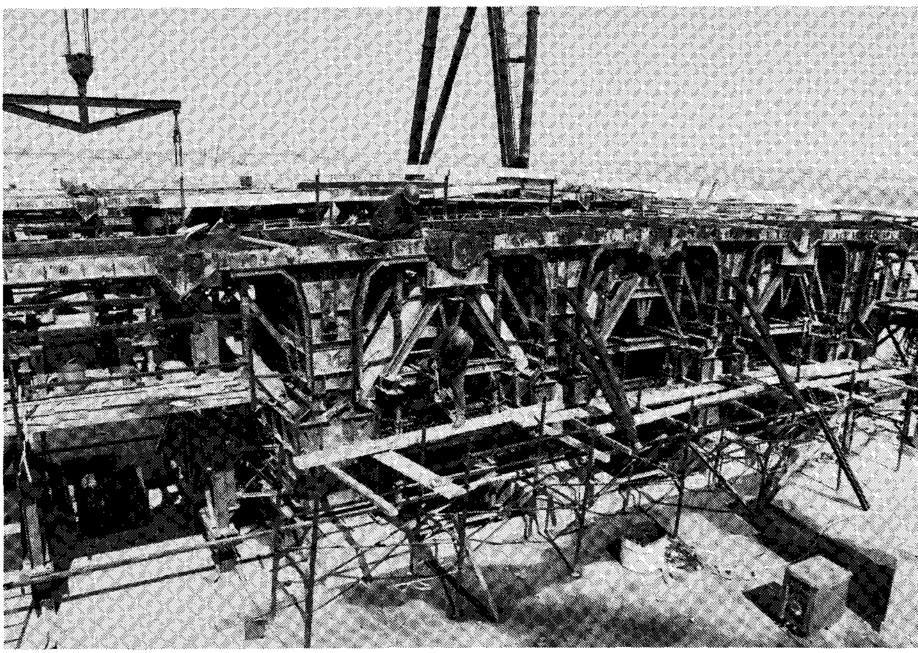


Fig. 22. Pier segment forms (courtesy of Bouygues).

Typical Segments

Typical segments are fabricated on three separate long-line beds,[4,9] which are of sufficient length to cast all segments required for a span. The fabrication beds are straight and horizontal. The longitudinal curved profile is obtained by means of a polygonal envelope, with breaks distributed on both sides of the pier segment, i.e., each span is a straight line chord of the profile. The geometry is obtained by adjusting the inclination of the pier segments at each end of the bed.

Soffit forms for the four longitudinal ribs are continuous for the entire span length bed. The form for the tendon trough at the bottom of the four ribs and those for the tendon outlets are mobile and relocate with the casting sequence (Fig. 23). Top and bottom flange soffit forms are of segment length and relocate in accordance with the casting sequence.

Precast triangles are positioned on jigs (see Fig. 24) which are then positioned in the casting bed prior to casting the bottom flange (Fig. 25). After casting and initial curing of the bottom flange, the bottom flange soffit form is transferred forward for the next segment; precast triangles are positioned and the bottom flange is cast for the new segment at the same time the top flange of the previous segment is cast. A casting cycle is illustrated in Fig. 26 and described as follows:

Day 1
1. Adjustment of Pier Segment A.
2. Forming bottom flange soffit, Segment 1.
3. Placing bottom flange reinforcement cage, Segment 1.
4. Positioning of precast triangles with longitudinal rib forms.
5. Installation of lower bulkhead form.
6. Removal of Segments 6 and 7 to storage.

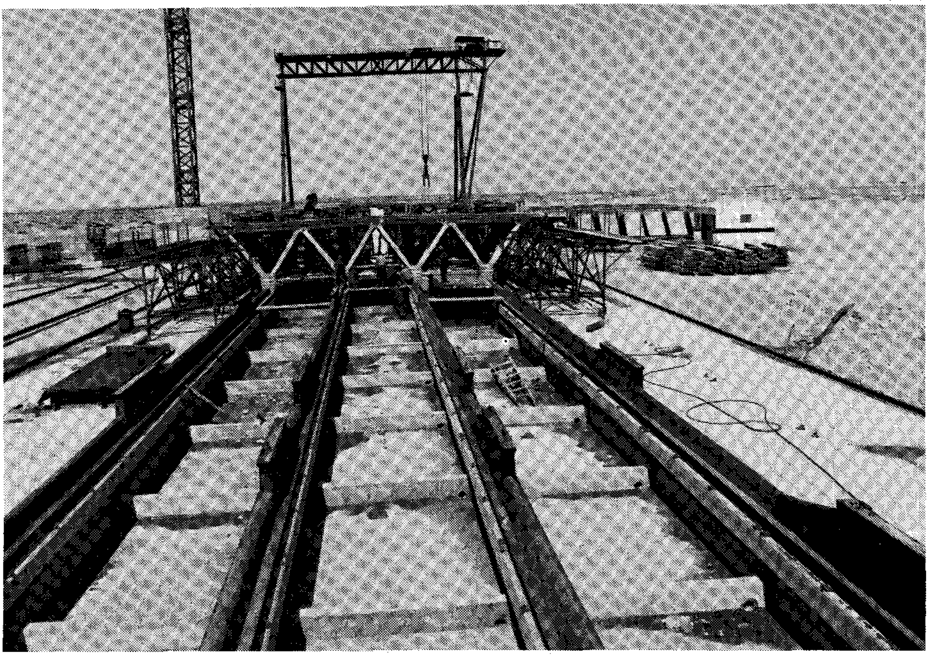


Fig. 23. Longitudinal rib soffit forms (courtesy of Bouygues).

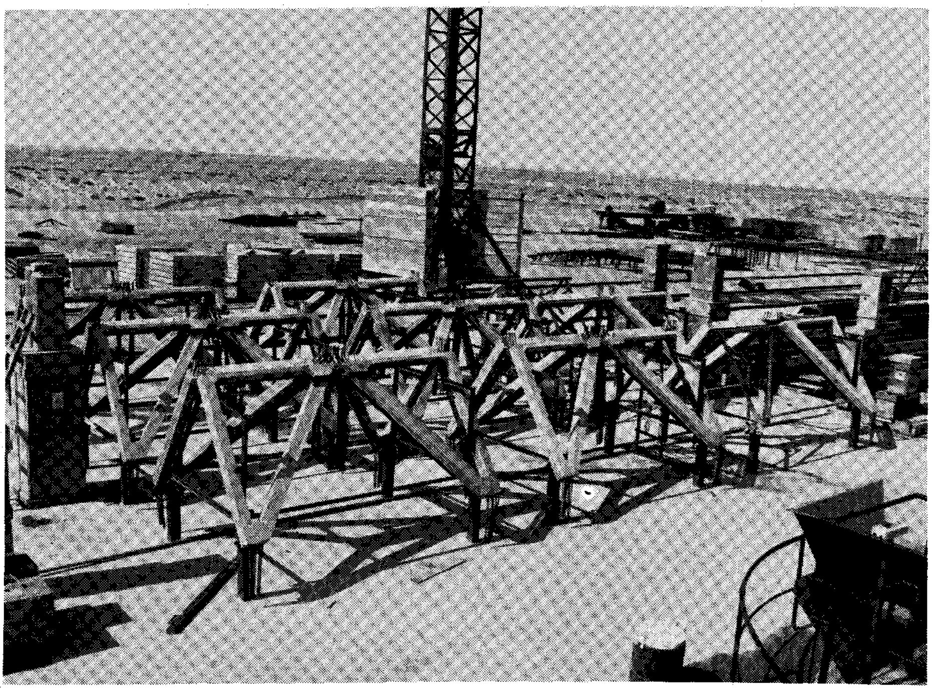


Fig. 24. Precast triangles positioned on jigs (courtesy of Bouygues).

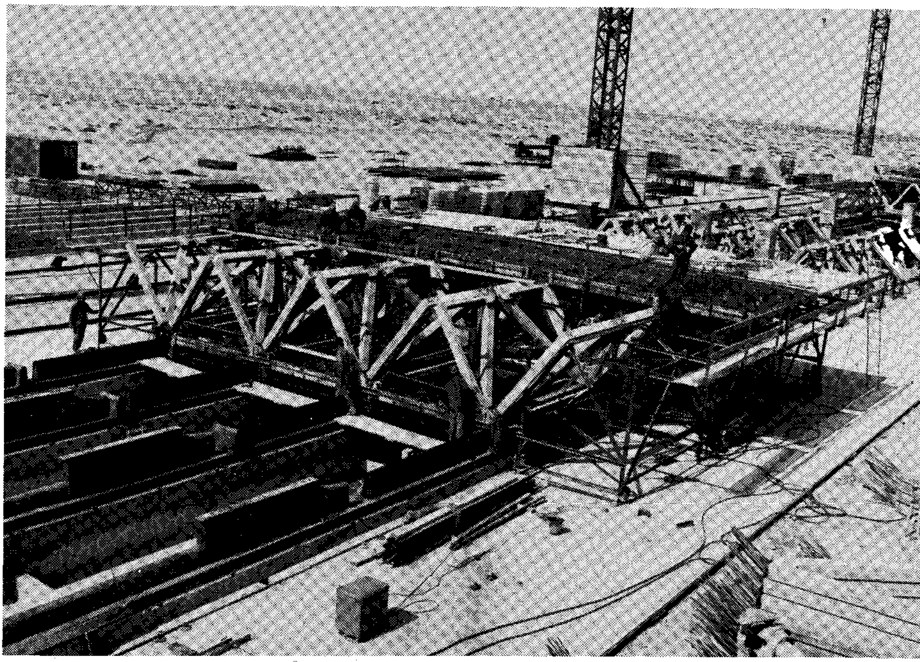


Fig. 25. Precast triangles on jigs positioned in casting bed (courtesy of Bouygues).

7. Concreting bottom flange of Segment 1.

Day 2
1. Stripping of bottom flange soffit form from Segment 1 and moving it forward into position for Segment 2.
2. Placing reinforcement for bottom flange, Segment 2.
3. Placing precast concrete triangles into position on Segment 2.
4. Top flange soffit form moved from Segment 8 to Segment 1.
5. Installation of lower bulkhead form Segment 2 and upper bulkhead form Segment 1.
6. Placing reinforcement top flange, Segment 1.
7. Segment 8 removed to storage.
8. Concreting bottom flange Segment 2 and top flange Segment 1.

Day 3
1. Stripping of bottom flange soffit form from Segment 2 and moving it forward into position for Segment 3.
2. Repeat Operations 2 and 3 of Day 2.
3. Stripping of top flange soffit form from Segment 1 and moving it forward into position for Segment 2.
4. Repeat Operations 5 and 6 of Day 2.
5. Concreting of top and bottom flange.

Day 4
1. Pier Segment A removed to storage.
2. Typical Segment 1 removed to storage with precast triangle jigs.
3. Repeat Operations 1 to 4 of Day 3.
4. Triangle support jigs of Segment 2 rolled backward.
5. Concreting top and bottom flange.

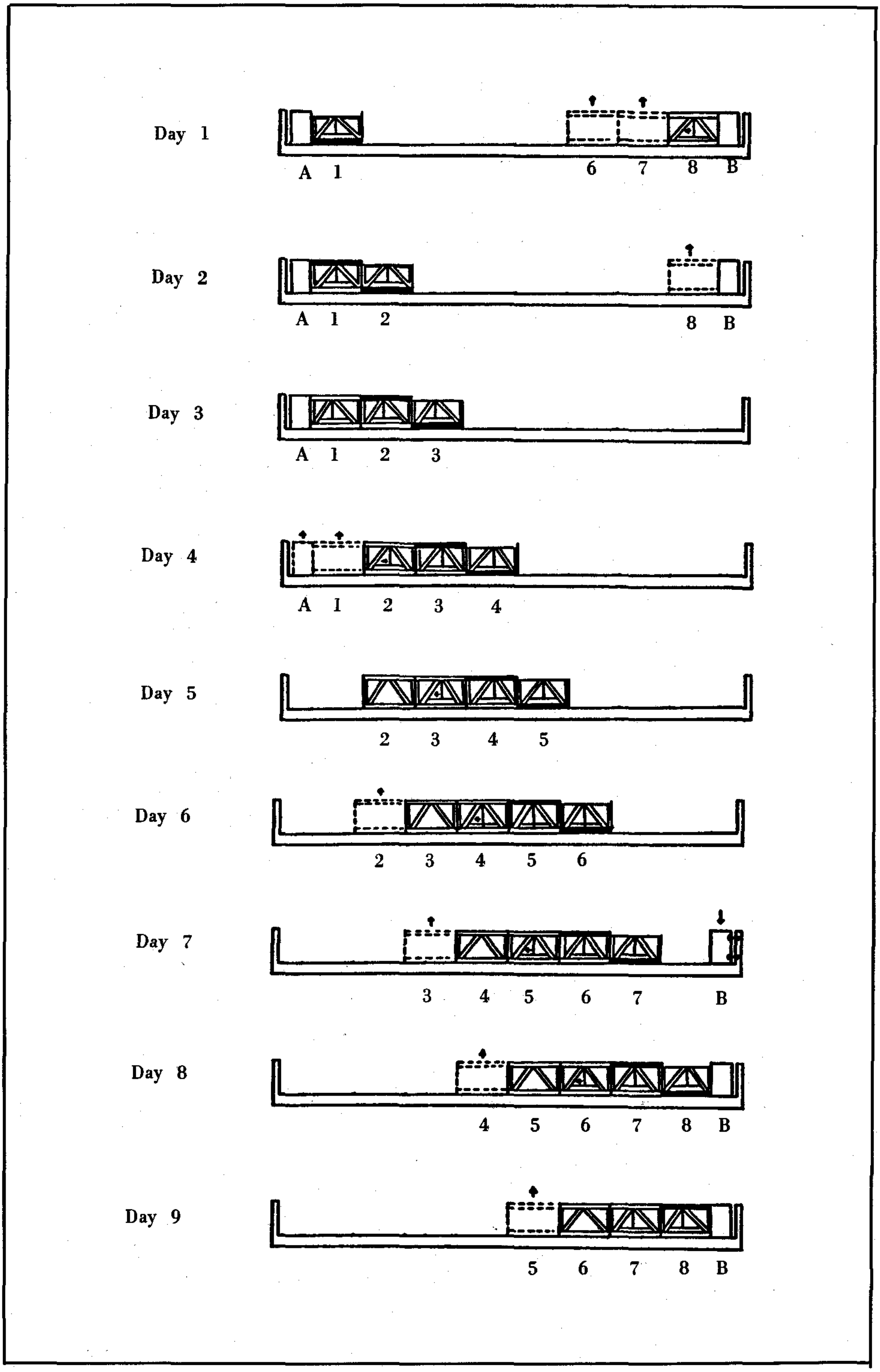


Fig. 26. Schematic casting cycle for typical segments.

Day 5 1. Repeat Operations 1 to 4 of Day 3.
2. Repeat Operation 4 of Day 4.
3. Concreting of top and bottom flange.

Day 6 1. Segment 2 removed to storage.
2. Repeat Operation of Day 5.

Day 7 1. Segment 3 removed to storage.
2. Repeat Operations of Day 5.
3. Placing and adjusting Pier Segment B.

Day 8 1. Segment 4 removed to storage.
2. Repeat Operations of Day 5, without bottom flange bulkhead.

Day 9 1. Segment 5 removed to storage.
2. Relocate top flange soffit form from Segment 7 to Segment 8, without bulkhead form.
3. Remove triangle jigs from Segment 7.
4. Strip bottom flange soffit form from Segment 8 and relocate to Segment 1 position.
5. Install top flange reinforcement in Segment 8.
6. Concreting of Segment 8 top flange.

ERECTION OF SUPERSTRUCTURE SEGMENTS

Erection of a typical span can be divided into three basic operations: (1) relocation of the launching gantry from the completed span to the span to be erected, (2) hanging all the segments in the span being erected from the launching gantry, and (3) post-tensioning operations.

The precast segments are transported from the storage yard to the launching gantry by trailers. The launching gantry is a cable-stayed steel structure that is capable of suspending all the segments in a span until they are post-tensioned and become self supporting. A trolley, moving below the launching girder, transports the segments from the trailer to its position in the span being erected.

The erection sequence for a typical span is schematically illustrated in Fig. 27, with the operations described as follows:

Step 1 — Post-tensioning of Span N is completed. The superstructure is supported on jacks at Piers N, N-1, and N-2. The clearance between the neoprene bearing pads and the pier segments at Pier N-2 is grouted with a non-shrinking grout and allowed to cure for 24 hours before stress is transferred to the bearings. Railway track, on which the launching girder moves, is positioned on top of the segments in Span N.

Sept 2 — The launching girder is moved forward on its wheels into position to erect the next span (N + 1). The central support (under the cable-stay pylon) is positioned on the centerline of the pier for a horizontal profile, or 16 cm (6 in.) forward or to the rear for a uphill or downhill grade of 3 percent, respectively. Excess post-tensioned strand length from Span N is cut to appropriate coupling length.

Step 3 — Jacks at the central support are engaged until the front wheels clear the track by 5 cm (2 in.). The portion of track over Pier N-1 is removed and the rear support is installed and jacked until the rear wheels clear the track by 5 cm (2 in.). The superstructure at Pier N-1 is adjusted by jacks to its proper geometry and the bearing pads grout packed with non-shrink grout.

Step 4 — The prestressing platform is removed from Pier N. The working platform at Pier Cap N is relocated to Pier N + 1. Installation and adjustment of jacks on Pier N + 1 is accomplished.

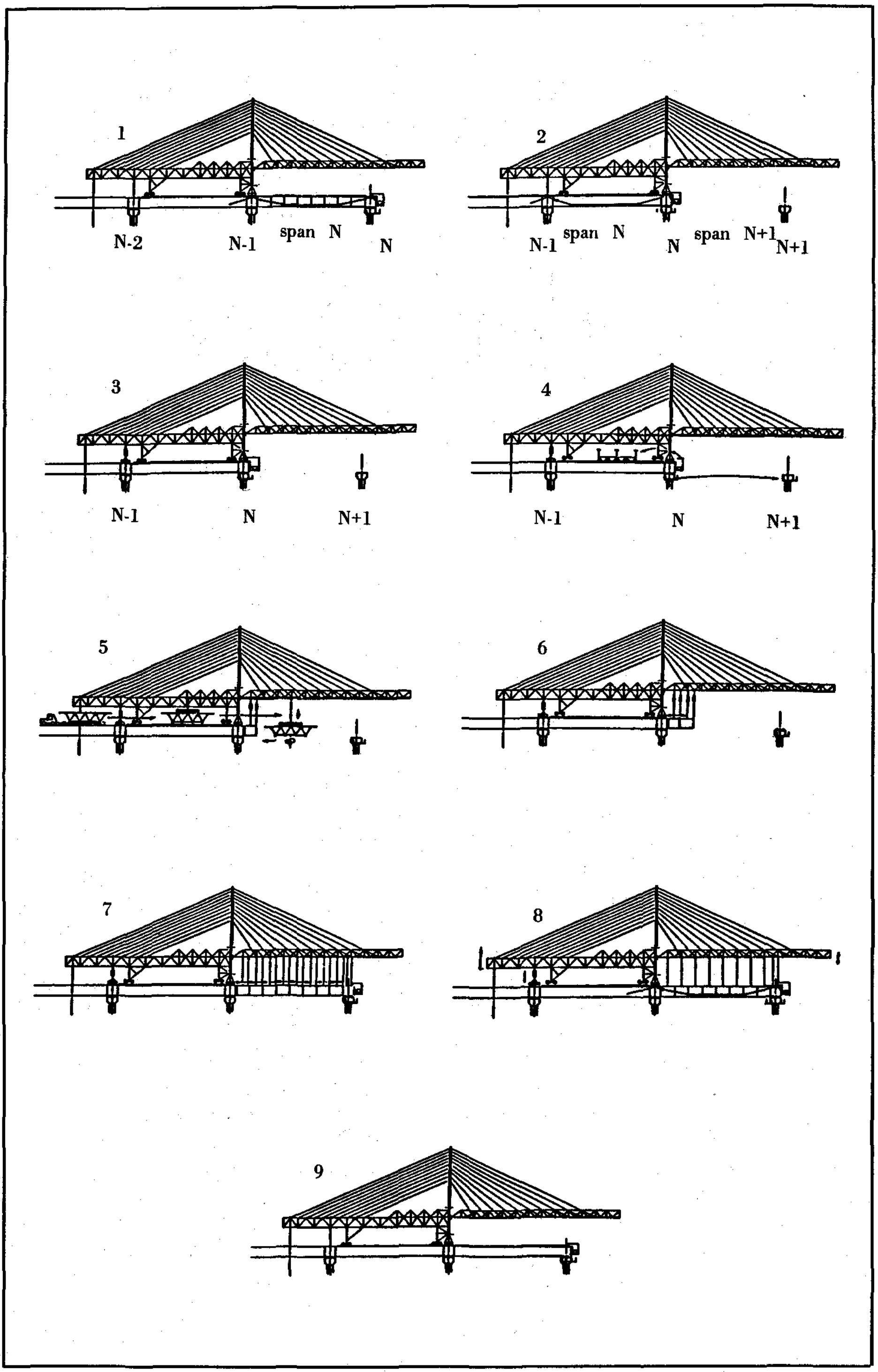


Fig. 27. Schematic erection sequence of a typical span.

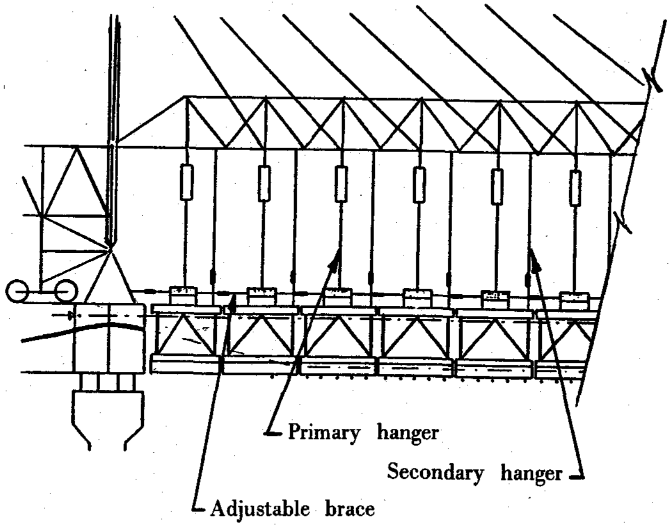


Fig. 28. Schematic of launching girder support hangers.

Step 5 — Typical Segment 1 is delivered by the trailer, picked-up by the launching girder trolley, moved forward and rotated 90 degrees, and positioned to within 20 cm (8 in.) of the pier segment. The segment is then supported by the primary and secondary hangers from the launching gantry (see Fig. 28). The trolley is released and two adjustable braces are positioned between the primary hanger and the central support. Segment 1 is then positioned to within 10 cm (4 in.) of the pier segment. The segment is then adjusted to its precalculated geometric position.

Step 6 — During the geometry adjustments of Segment 1, Segment 2 is delivered and Step 5 operations are repeated for Segment 2.

Step 7 — Remaining segments are positioned as described above. The geometry of the deck at Pier N is checked and adjusted if necessary. The prestressing platform is positioned at Pier N+1. Span N+1 is positioned and the rear support is adjusted. Ducts and tendons are installed.

Step 8 — Tendons are tensioned in pairs and the stress in the rear support is adjusted after each tensioning phase. During this operation the neoprene bearings at Pier N-2 are placed under load and the jacks removed. Upon completion of stressing operations, the elevations of the pier segments at N and N+1 are checked and adjusted. The hangers are released by jacking up the rear support. The launching girder is then set down on its rear wheels.

Step 9 — All hanging equipment is removed. The launching girder is set down on its forward wheels. Track is laid in Span N+1. The cycle is repeated for the next span.

In Step 5 above and Fig. 28, the two primary hangers are used to adjust the elevation and the transverse geometry and the two secondary hangers are used to adjust the longitudinal inclination. The horizontal adjustable braces allow positioning of the segment with the previously erected segment. The position of each hanger is such that there is a clearance of approximately 10 cm (4 in.) between the first typical segment and the pier segment. Thus, the launching girder is allowed to take its bending strain under full load of all the

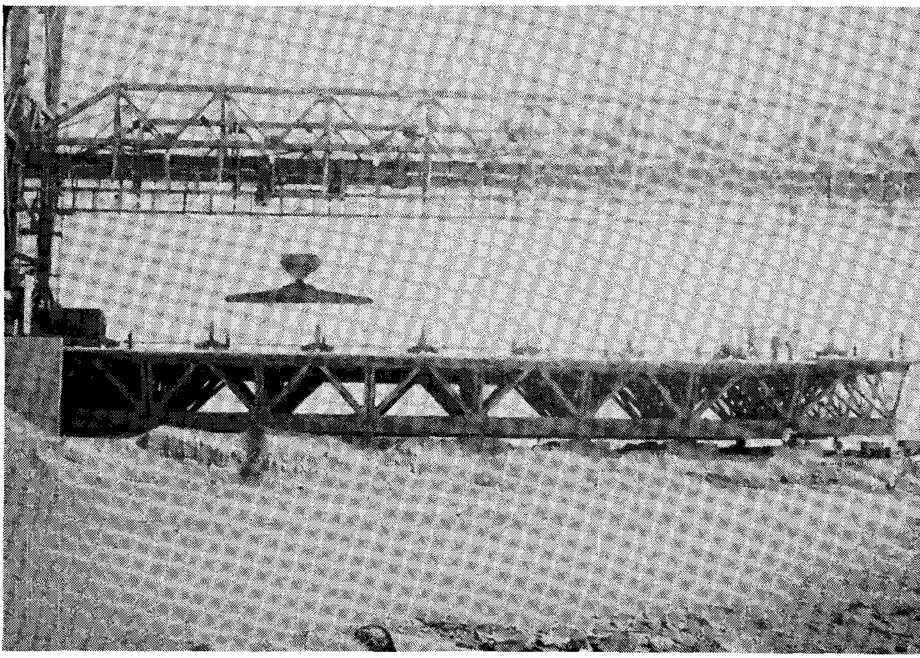


Fig. 29. Erection of first span of bridge. Note that embankment under the first span is temporary and used only as a precautionary measure.

segments without colliding with the pier segment.

Similarly, the same gap is maintained between segments during transfer of load from the launching gantry trolley to the primary hanger. The gap is then closed up by the horizontal braces before post-tensioning. The primary hanger is equipped with a pressure regulator to limit the transfer of load between hangers during the post-tensioning operations.

Post-tensioning is accomplished by stressing the tendons in pairs. The superstructure load is gradually transferred to the temporary supporting jacks by adjusting the stress in the rear support of the launching girder. The jacks are coupled together so as to balance the support reaction. Upon completion of prestressing operations, the clearance between neoprene bearings and the pier segments is grouted with a non-shrink grout and allowed to cure for 24 hours before transferring the reaction from the temporary jacks to the permanent bearings.

Since the navigation span is three segments longer, a modification of the above procedure is required. The first three segments are erected by the conventional cantilever method from the preceding span rather than being suspended from the launching girder. In order to preclude imbalance of the launching girder, a forward support is provided for the launching girder at the forward pier. The balance of the segments comprising the navigation span are then erected following the procedure outlined for a typical span.

Figs. 29 and 30 show the erection of the first span. The embankment under the first span is temporary and was used by the contractor as a precautionary measure during "debugging" of the launching girder and adjustment of the stays.

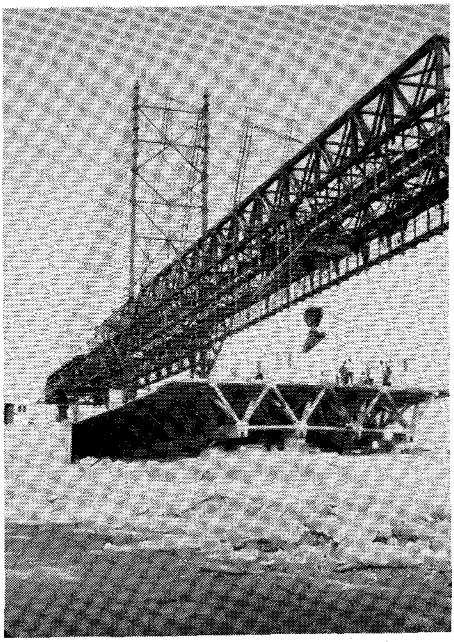


Fig. 30. Front view of erected first span.

MODEL TESTING

Bouygues, upon receiving a turnkey contract for the Bubiyan Bridge project, obtained the cooperation and services of the French Ministry of Transport for the designing and testing of a model (see Fig. 31) that would be representative of the 3-D superstructure concept of the Bubiyan Bridge.

The French Ministry of Transport was very interested in participating, because of the obvious future implementation of this concept in France, and delegated several agencies to the model testing activities. These organizations and their responsibilities are defined as follows:

1. Service d'Etudes Techniques des Routes et Autoroutes (SETRA), (Roads and Highways Technical Studies Department), who were responsible for the computational aspects of the tests using the STRUDL program.

2. Laboratoire Central des Ponts et Chaussées (LCPC), (Central Laboratory for Roads and Bridges), who defined and mounted the instrumentation and were responsible for data acquisition.

3. Direction Regionale de l'Equipment (DRE), (Regional Infrastructure Department), who were responsible for the testing and control of the operation.

4. Bouygues was responsible for the design and construction of the test model.

The purposes of the test model were to determine the behavior and stress of the diagonals and its correlation with the design, to verify the design with respect to any movement in the dry joints of the segments, and to determine the coefficient of friction of the tendons at the points of tendon profile deviation.

A comparison of the cross section of the typical segment for the Bubiyan Bridge and the test model is presented in Fig. 32. Typical segment length, height, and other dimensions of the model are consistent with those of the prototype with the exception of the overall width of the segment and the bottom flange thickness. Since there are three bottom flange elements in the prototype and only one in the model, the thickness of the bottom flange in the model was increased to preserve the same center of gravity of the cross sections.

In elevation (Fig. 33) the model has an approximate span between supports of 25 m (82 ft) and a cantilever of approximately 8 m (26 ft) to simulate continuity of superstructure at a pier. Because of the differences in the model and prototype, additional dead weight was suspended from the model such that dead load stresses in the model were representative of those in the prototype.

Live loads were introduced into the structure by tensioning bars attached to the top flange and jacking against a raft foundation below the model. These tensioning bars were located at nine appropriate locations in the super-

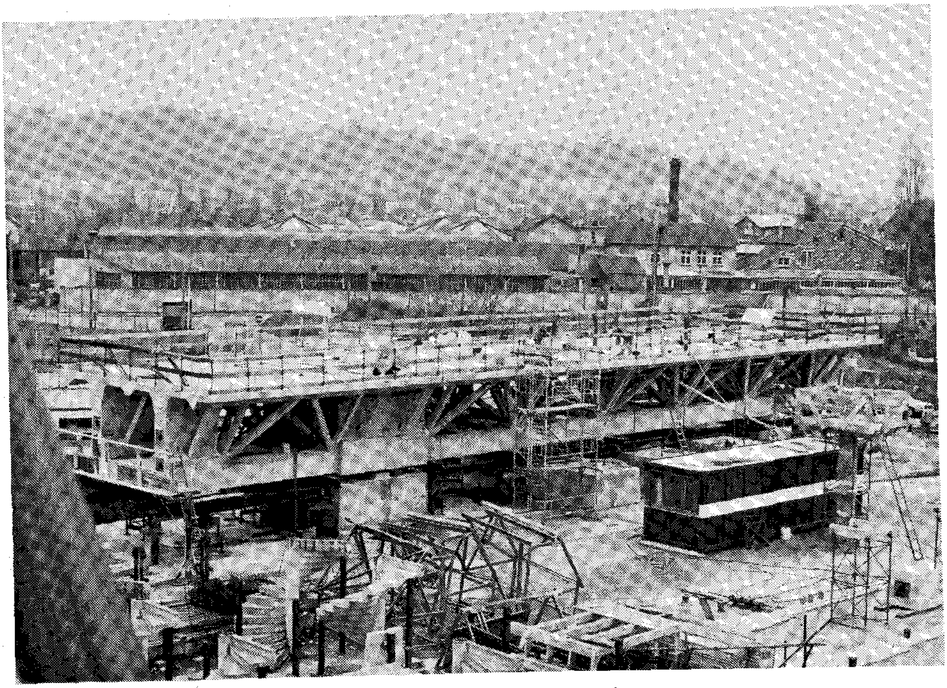


Fig. 31. Overall view of test model (courtesy of Bouygues).

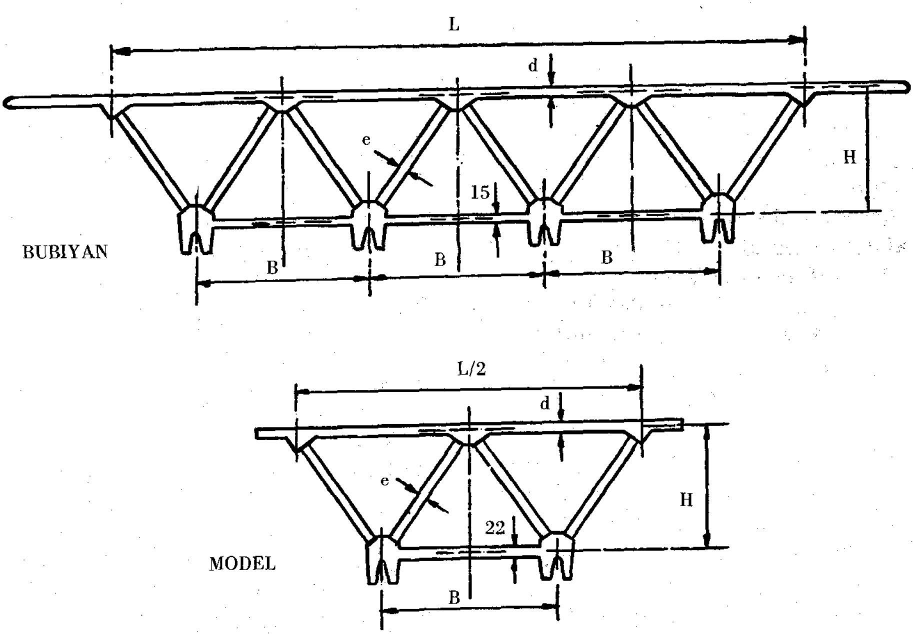


Fig. 32. Comparison of test model and Bubiyan Bridge cross section.

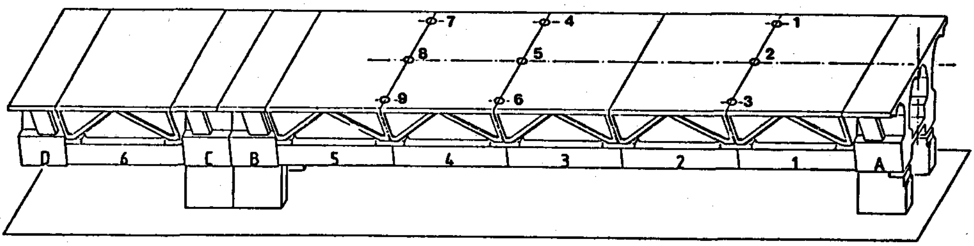


Fig. 33. Isometric of test model showing location of tensioning bars.

structure (Fig. 33). The location of the live load inducing tension bars also allowed the introduction of transverse symmetrical and unsymmetrical loading.

The longitudinal prestressing of the model consists of four tendons, two per longitudinal bottom rib, consisting of 24 T 15 tendons (24 strands of 0.6 in. diameter). The prestressing force in the model is approximately 10 percent less than that of the prototype such that the ratio of unit prestress force to shear stress is representative of that in the prototype to determine slip in the dry segment joints.

Preliminary and partial evaluation of short-term loading indicates good agreement with design. Vertical deflections of the model confirm design values. Cracking was observed only in tension diagonals with the order of occurrence complying with design assumptions. Cracks were regularly spaced and essentially corresponded to the spacing of tie reinforcement [20 to 40 cm (8 to 15 in.)]. Crack width was a maximum of 0.15 mm (0.006 in.) and a mean of 0.07 mm (0.0028 in.) at 100 percent loading. Loading for this case was in increments of 10 percent with the load being taken to zero between loading steps. Behavior of the diagonal node points was unaffected by the cracking and all cracks closed when the elements returned to a compression state from a tension state.

Measured friction coefficient in the tendons was 0.24 as compared to a value of 0.3 used for the design of the Bubiyan Bridge.

No slippage was observed in the match-cast segment dry joints.

The above data are based upon a partial evaluation of data from the short-term loading tests. Analysis of the remaining data is continuing. Upon completion a determination will be made as to any additional testing that may be required.

Anticipated future testing includes (1) long-term testing of from 3 to 4 months with data acquisition and visual inspection at approximately 2-week intervals, (2) vibration testing whereby a vibrator will be mounted at various points on the deck and thus provide data regarding the period of vibration in the diagonals, (3) temperature effects to be gathered continuously over a 2-day period of substantial sun exposure, and (4) test to failure.

MODIFICATIONS TO STRUCTURE FOR BUBIYAN ISLAND RAMP

It was recognized during the design stage that a serious soils consolidation problem existed on Bubiyan Island. To avoid the anticipated long period of significant and unacceptable settlement of the embankment ramp approach to the bridge on the Island, which would have resulted in future high maintenance costs, it was decided to add sand drains to achieve rapid consolidation of

NOTE:
For units V1 through V3 see Fig. 6

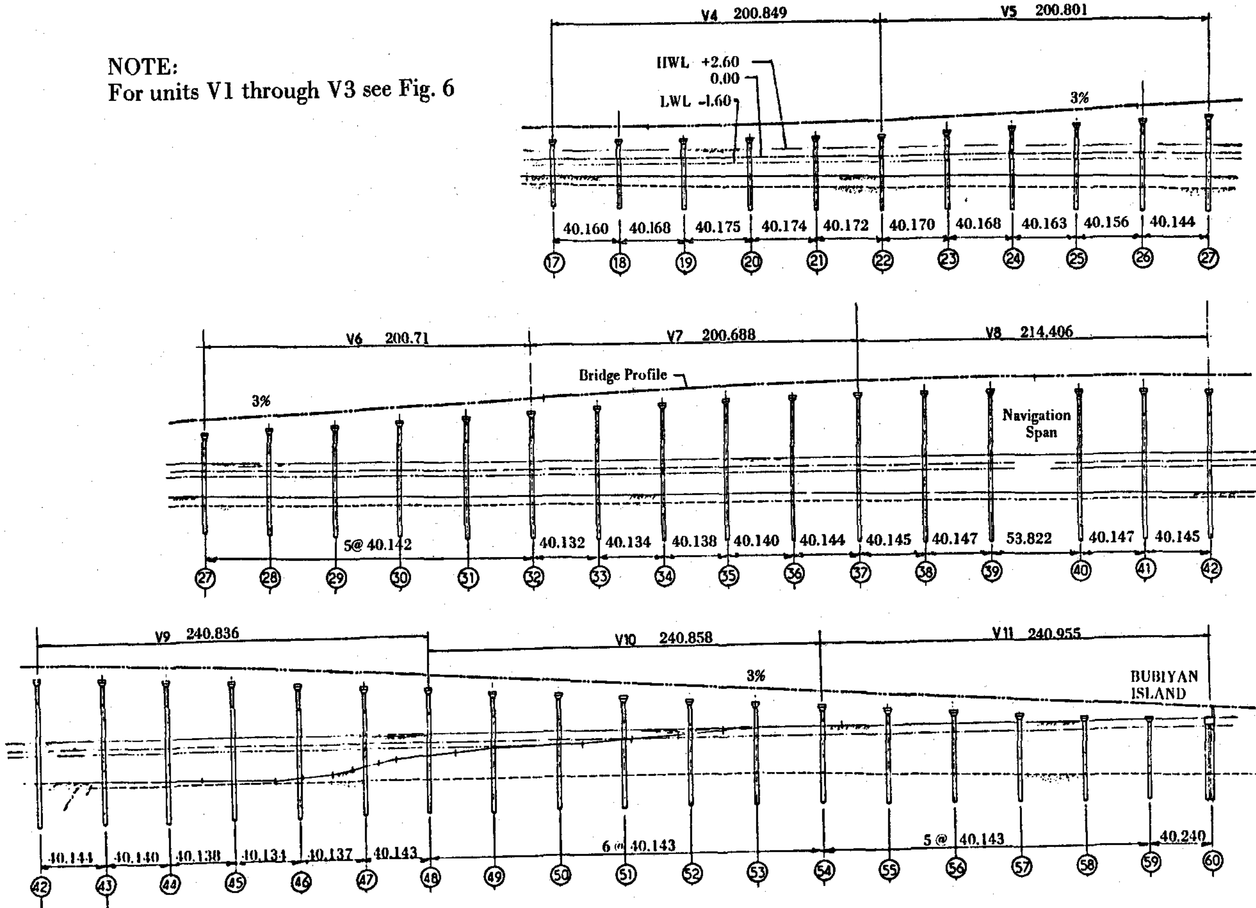


Fig. 34. Adjusted profile of Bubiyan Bridge.

1m=3.2808 ft

Table 4. Adjusted tabulation of spans, Bubiyan Bridge. Dimensions are in m (ft).

Unit	Span dimensions in unit	Unit length
V1	6 @ 40.16 (131.7585)	240.96 (790.5511)
V2	5 @ 40.16 (131.7585)	200.80 (658.7927)
V3	5 @ 40.16 (131.7585)	200.80 (658.7927)
V4	40.16 – 40.168 – 40.175 – 40.174 – 40.172 (131.7585 – 131.7848 – 131.8077 – 131.8045 – 131.7979)	200.849 (658.9534)
V5	40.17 – 40.168 – 40.163 – 40.156 – 40.144 (131.7913 – 131.7848 – 131.7684 – 131.7454 – 131.7060)	200.801 (658.7959)
V6	5 @ 40.142 (131.6995)	200.71 (658.4974)
V7	40.132 – 40.134 – 40.138 – 40.140 – 40.144 (131.6667 – 131.6732 – 131.6864 – 131.6929 – 131.7060)	200.688 (658.4252)
V8	40.145 – 40.147 – 53.822 – 40.147 – 40.145 (131.7093 – 131.7159 – 176.5814 – 131.7159 – 131.7093)	214.406 (703.4318)
V9	40.144 – 40.140 – 40.138 – 40.134 – 40.137 – 40.143 (131.7060 – 131.6929 – 131.6864 – 131.6732 – 131.6831 – 131.7028)	240.836 (790.1444)
V10	6 @ 40.143 (131.7028)	240.858 (790.2166)
V11	5 @ 40.143 – 40.24	240.955 (790.5348)

Total length: 2382.663 (7817.1358)

the ramp structure. This solution then allowed a reduction in structure length (from that indicated in Fig. 6 and Table 2) by three spans on Bubiyan Island, which therefore allowed an economic trade-off resulting in no additional cost to the project (see Fig. 34 and Table 4).

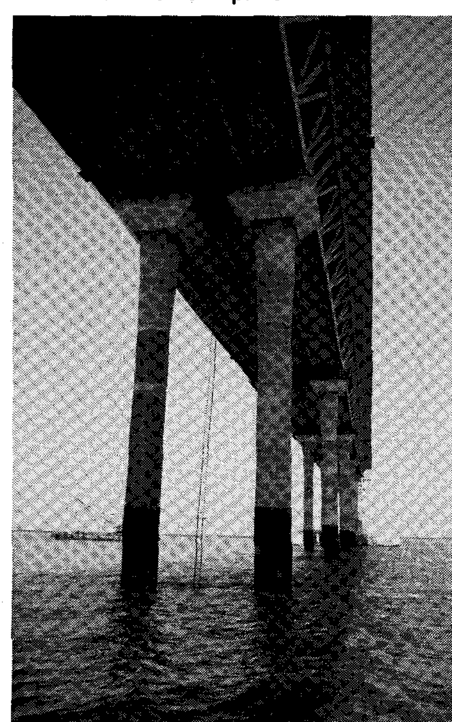
To minimize the impact of this revision on the overall bridge profile, the location of the main span for the navigation channel was relocated approximately 200 m (660 ft) closer to the mainland without seriously affecting navigation.

Although pier construction and segment casting were already underway, the above revisions were implemented before they had any impact upon work already completed, and without any delay to the progress of the project.

ERECTION

Erection of the superstructure has proceeded smoothly and according to schedule. Figs. 35 through 37 show various stages of erection of the superstructure.

Fig. 35. Longitudinal view of underside of superstructure. At this stage bridge is about two-thirds complete.



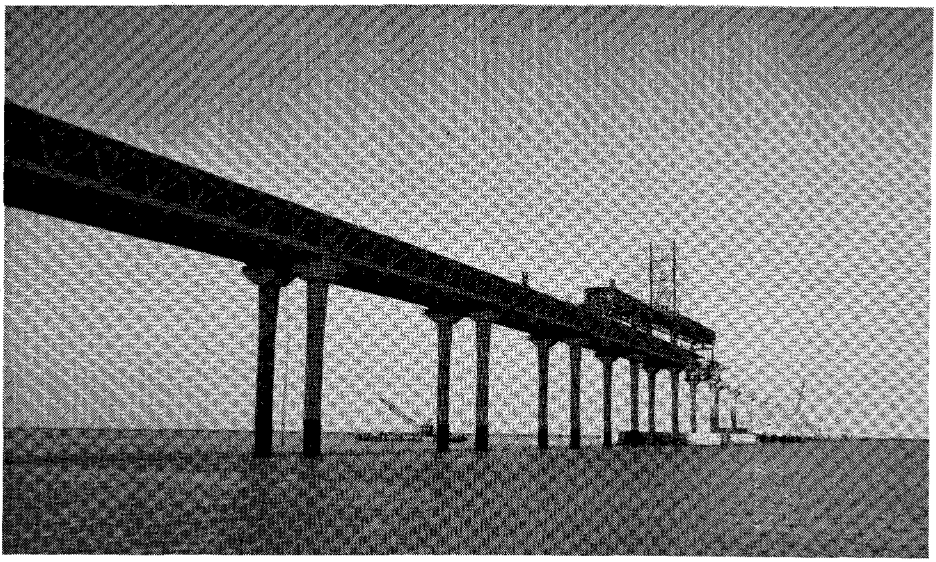


Fig. 36. Longitudinal view of superstructure during erection.

Fig. 37. Typical precast truss element being lifted into position.

In these photographs, the erection has reached the navigation span. The bridge is approximately two-thirds completed.

The superstructure was essentially completed by the end of December. It is estimated that the entire project will be finished by April 1983.

CLOSING REMARKS

The Bubiyan Bridge project represents a significant step forward in the state-of-the-art of bridge construction and particularly in precast prestressed segmental bridge construction. In this paper the authors have attempted to bring to the attention of the profession the significant aspects of the design, model testing, fabrication and construction of the Bubiyan Bridge with the intent of encouraging and stimulating others to consider the concept of three-dimensional space frame bridge structures.

The Bubiyan Bridge not only represents an important and significant accomplishment in the transportation network of Kuwait, but more importantly a new and higher level in the technology and art of constructing bridges.

The innovative design and construction procedures expressed in the Bubiyan Bridge are such that not only Bouygues and others associated with the project, but the entire fraternity of those concerned with bridge design and construction world wide, can justifiably take a great deal of pride and sense of accomplishment.

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