特別講演Ⅰ

CANADIAN BRIDGE CROSSING A THIRTEEN KILOMETRE STRAIT
（カナダ連邦橋に関する和文概要）

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1.はじめに

カナダ東端のプリンスエドワード島（PEI）州とカナダ本土のニューブランズウィック（NB）州との間にあるノーサンバーランド海峡を横断する12,910mの橋梁が1997年5月31日に開通した（Fig.1参照[3)]）。この橋梁は、次の2つの異なる方法により架設された；

(a) アプローチ橋部は、3mのプレキャストセグメントをラウンチングエレクショントラスにより張出し架設する。

(b) 主橋部は、各径間が250mであり、4つの大きなプレキャスト部材を大型フローティングクレーンにより運搬、架設する（Fig.2b）。

どの径間においても上部工は、Table 1に示すような変断面1室箱桁である。Fig.3aは、各スパン250mの主橋部における3径間連続構造の部分を示す。その構造は、門型フレームGCDHと2つの張出し桁BC及びDEから構成される。各スパンは、2つの中間ヒンジ（A、B及びE、F）を有し、ヒンジ部で伸縮装置が施される。桁AB及び桁EFは60mであり、両側の門型フレームから張出した桁に載る。

2.設計基準

2.1　荷重係数と抵抗係数

カナダ建設省は本橋の設計基準を定め、一般の橋梁に対するよりも厳しい次のような要件を設定した。

1. 耐用年数　100年
2. 荷重係数と抵抗係数は、十分定評ある確率及び信頼性理論に基づいた安全係数により決定しなければならない。例えば、100年寿命の場合、終局限界状態に対する安全係数は、$\beta = 4.0$である。
3. 本橋は、中央付近において、幅172m及び高さ49mの航路部を有する。

2.2　風荷重と潮流荷重

橋軸直角方向の10分間平均風速として、平均海上高さ10mで26.5m/sを考慮する。この数値は、オンタリオ州ロンドンのBLWTLにおける風洞実験から得られたものである。この模型実験は、架設時と完成系について行われた。主橋部ピア上の上部工完成系に作用する等価横風力は、3.7MNである。

最大潮速は2.0m/s、海峡中央部における最大波高は4.0m、ピアベースに作用する合波力は、水平、鉛直（下方）及び転倒モーメントに関して次表のとおりである。

水深（m）	水平力（MN）	鉛直力（MN）	転倒モーメント（MN·m）
16	8.0	4.3	53.4
25	8.1	3.5	118.5
35	8.2	2.7	192.6

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Gamil Tadros and Amin Ghali: Canadian Bridge Crossing A Thirteen Kilometre Strait

2.3　氷塊荷重

ノーサンバーランド海峡では、流氷が 12 月から 3 月までみられる。潮流、波及び風により氷が移動し、氷脈が形成される。流氷の最大厚さは 2.3m、また、氷脈の最大高さは 2.5m に達する。ピアの設計に適用する氷圧力とその経時変化及び Fig.4 ～ 6 のアイスシールドの諸元を決定するために膨大な理論的、実験的研究が行われた。ピアは、水面部で水平から 52 度の傾斜した円錐である。氷がピアに作用するとき、流氷は、傾斜面を上がると同時に、自重で曲げ破壊する。このような破壊性状の場合、ピアに作用する力は、氷を鉛直方向に破壊させる力より大幅に小さくなる。設計に使用する氷圧力の水平成分の値は H ＝ 16.4MN（安全係数を乗じると 30.0MN）である。

3.　アプローチ橋部

アプローチ橋部は比較的浅い水深の場所にあるので、主橋部の架設に用いる大型フローティングクレーンを使うことができない。そのため、アプローチ橋部（各スパン長 96m）の架設には、従来のプレキャストカンチレバーセグメント工法が用いられる。代表的なバランストカンチレバーとピアを Fig.10 に示す。桁高 3.6m ～ 5.2m のプレキャスト矩形箱桁セグメントがポストテンション方式のケーブルで一体化される。30 のプレキャストセグメントから構成されるバランストカンチレバーは、ラウンチングエレクショントラスにより架設される。また、伸縮継手が桁中央に設けられ、この伸縮継手部にはスライド可能な鋼桁が挿入される。鋼桁は、カンチレバー端部で橋軸方向の動きのみを許容し、横方向の動き及び回転は拘束する (Fig.10*)。

4.　主橋部

主橋部は 45 径間であり、その内の 43 径間がスパン長 250m である。各スパンは、4 つの大きなユニットから構成される。各ユニットはヤードで製作され、海峡の所定の位置でスワン（Svanen）と呼ばれる強力なフローティングクレーンを用いて据え付けられる。ユニット数を少数にしたのは、海洋における場所打ちコンクリートや PC テンドンによるユニットの接合作業を最小にするためである。4 つのユニットというのは、Fig.4 に示すピアベース、ピアシャフト、主桁ユニット及びドロップイン桁ユニット（吊り桁用と連続桁用）であり、各ユニットを以下に説明する。

4.1　ピアベース

このユニットは、海床面から平均海面上 4 ｍの高さに位置し、4 つのセグメント、つまり、フーチングリング、円錐フーチング、水深により変化するフーチング円筒部そしてフーチング上部からなる。フーチングリングの外径は 22 ｍまたは 28 ｍのいずれかであり、大きい方は、水深 27 ｍ以上に位置する 8 基のピアに対するものである。どの径のフーチングリングにも、同じ大きさの円錐フーチングが取り付けられる。ピア位置での水深の変化に対しては、フーチング円筒部の高さの調整により対応する。フーチング上部はアイスシールドを受ける柱状構造であり、この構造は全てのピアに設けられている。

ピアベースユニットの総重量は、3500 ～ 5200 ｔであり、海峡の所定の位置まで運搬され、予め水平調整された 3 点のフーチングサポートに据え付けられる。このフーチングサポートは、プレキャストコンクリートであり、海底をリング状に掘削した後に設置される。フーチングリングと掘削底面の間には、水中コンクリートが充填される。

4.2　ピアシャフト

ピアシャフトユニットはアイスシールドとピアシャフトから構成される（Fig.11）。アイスシールドはピアベースの柱頭部に適合する截頭中空円錐である。この円錐は、平均海面上 4 ｍ～ 5 ｍで下方へと広がっている。円錐の外径は底面で 20 ｍである。円錐の母線を水平軸と 52 度の角度としたアイスシールドには、コンクリートの強度が 55　MPa のものが用いられ、また、中間の 2 ｍの部分は特に強度が 100　MPa のハイパーフォーマンスコンクリートが用いられた。
ピアシャフトは、上面外部で 5m × 10 ｍの中空矩形断面から底面外部で一辺 8 ｍの中空八角形断面へと変化する。シャフトの部材厚は、600 ｍｍである。ピアシャフトユニットの重量は、

大きいもので 4000 t である。ピアシャフトとピアベースは、PC テンドンとグラウトにより接合される。

4.3 主桁ユニットとドロップイン桁ユニット

主桁ユニットは長さ 192.5 mの箱形断面であり、桁高は、ピア上の 14 mから支間中央の 5 mまで変化する。主桁ユニットの重量は 7800 t であり、ピアシャフトユニット上に据え付けられ、エポキシ樹脂とポストテンションテンドンにより接合される。張出し長はピアの中心線から先端部まで 97.5 m及び 95 mである（Fig.4）。ピアシャフト位置で主桁を受け、マッチキャスト接合するが、主桁ユニットの正確な位置決めを容易にするためにコンクリート版が挿入される（Fig.6,11）。

主桁ユニットは、バランストカンチレバー法により製作ヤードで作られた 17 ブロックのセグメントからなる。ピアセグメントは、長さが 17 mで逆 V 字形の鋼製ダイアフラムで補強されたブロックであり、主桁とピアシャフト間のモーメント差を受けるために用いられる（Fig.6）。他の 16 のセグメントの長さは、7.5 mから 14.5 mまで変化するが、その容積はほぼ等しい（Fig.11）。

張出しが 97.5 mの桁の先端部において、連続桁用ドロップイン桁が場所打ちコンクリートとポストテンションテンドンにより接合される。このドロップイン桁ユニットは、桁長 54 mであり、両側に張出し桁を有する門型フレーム構造とするために、両方の桁の端部に 0.5 mの閉鎖形場所打ちジョイントで接合される。このユニットの重量は 1200 t である。

一方、張出しが 95 mの桁は、その先端部で吊り桁用ドロップイン桁を受けるようにコーベルの形状をしている。この吊り桁用ドロップイン桁は、桁長 60 mであり、コーベル上に支持される単純桁である。このドロップイン桁ユニットの重量は 1600 t である。

主桁の高さの変化をみると、桁下面の曲線がスパン中央で 4.5 m、ピア上で 14 mの値をとるパラボラとなるように設計されている（Fig.2a）。

5. 施工法

橋梁ユニットは、ヒュースマン（Huisman）と呼ばれる運搬車（Fig.12、オランダの Huisman Iterc 社製）により、プレキャストユニット製作ヤード内を移動し桟橋まで運ばれる。このヒュースマンは、大きな重量のユニットを持ち上げたり、移動させたりできる背の低い油圧式の台車である。ヒュースマンには、ステンレス鋼板で覆われたコンクリート軌道上をスライドできるようにテフロン版とステンレス鋼ベルトが付いている。

スワンと呼ばれる双胴船に付けられた重量クレーンにより、全てのユニットを桟橋で吊り上げ、海峡に運搬し、所定の位置で建て込む（Fig.13 ～ 16）。海峡におけるユニットの施工に必要な海洋工事は、次の章で述べる。

6. 海洋作業

主橋部の架設においては、次の海洋作業が必要である。

ステップ 1：ピアの位置で表土の厚さ、基盤の強度及び基礎の高さを決定するための地質調査を行う。

ステップ 2：掘削機械を用いて、ステップ 1 で決定した基礎の位置において表土 2 mを取り除く。

ステップ 3：ステップ 2 で掘削した場所の中心に鋼円盤を置き、その周辺にピアのフーチングリングを設置するための幅 6 mのリング状トレンチをつくる。このトレンチは、基礎の位置まで掘り下げられる。

ステップ 4：トレンチを清掃した後、直径 5 mのプレキャストコンクリート製パッドを設置する。その設置誤差は、x、y、z の 3 方向の許容値を満足する必要がある。これらのパッドは、ピアベースの仮受け台となる。

ステップ 5：スワンによりピアベースユニットを運搬し、ステップ 4 で作製したコンクリートパッドの上に置く。

ステップ　6：ピアベースユニットを据え付け、その許容誤差をチェックした後、フーチングリングの下の支持力が均等となるように 55m^3 のトレミーコンクリートを打ち込む。

ステップ　7：トレミーコンクリートが所定の強度に達した後、スワンによりピアベースの柱頭部にピアシャフトを据え付ける。柱頭部にセットしたジャッキを使いピアシャフトユニットが正しい位置にくるように調整する。柱頭部とピアシャフトの間をグラウトにより密着させる。グラウトが所定の強度に達した後、PC テンドンによりピアベースとピアシャフトが一体化される。

ステップ　8：主桁ユニットの据え付けを正確なものとするためのガイドとしてピアシャフトの上面にコンクリート版を設置する。コンクリート版の位置を厳密に調整した後、コンクリート版とピアシャフトの上面間にグラウトを注入する。グラウト硬化後、デビダーグ（Dywidag）PC 鋼棒によりコンクリート版とピアシャフトを一時接着させる。

ステップ　9：スワンを使い、エポキシ樹脂が塗布されたコンクリート版上に主桁ユニットを載せ、主桁ユニットの下面をコンクリート版のマッチキャストシアキーに合わせる。ピアシャフトの上部に据えられた水平ガイドと垂直ジャッキを使い、主桁ユニットの振動を抑える。ステップ 9 が終了すると T 型構造の完成となる（Fig.21）。

ステップ 10：ステップ 1 ～ 9 を繰り返し、次のピアでもピアベースユニット、ピアシャフトユニット及び主桁ユニットよりなる T 型構造を製作する。

ステップ 11：このステップは、完成した 2 つの T 型構造の間に連続桁用ドロップイン桁を接合することである。スワンにドロップイン桁ユニットを載せ、張出し桁端部の上面の適当な位置に突起した鋼支承を付ける。徐々に、ドロップイン桁ユニットを下すとともに主桁ユニットとの相対的動きを制御するために水平と垂直のジャッキを用いる。桁が下りると、2 つの張出し桁の先端部は、それぞれ変位しようとするが、水平ジャッキがこの動きを拘束するように作動し、ドロップイン桁ユニットと張出し桁に圧縮力を生じさせる。ドロップイン桁ユニットと主桁ユニットの間にコンクリートを打ち込み、さらに PC テンドンで 3 つのユニットの連続性を完全なものにする。ステップ 11 が終了すると両側に張出し桁を有する門型フレーム構造の完成となる。

ステップ 12：ステップ 1 ～ 11 を繰り返すとともに、吊り桁用ドロップイン桁ユニットを架設して、主橋部の完成となる。

海洋作業と平行して、橋面工が施工される。これは、伸縮装置、安全地帯、照明施設、料金所及び交通標識の設置などである。

7．結び

本稿は 13 Km の海峡を横断するコンクリート橋に関するものであり、設計基準及び架設法について紹介した。本プロジェクトは、本契約調印後、44 ヶ月で完成した。積極的なプレキャスト化、プレキャストユニット製作ヤードの綿密な設計（Fig.17 ～ 20）、そして大型のプレキャスト部材の運搬と作業に用いた装置などにより、海洋工事が最小となるよう配慮されている。

CANADIAN BRIDGE CROSSING A THIRTEEN KILOMETRE STRAIT

Gamil Tadros[1] and Amin Ghali[2]

1. INTRODUCTION

The Northumberland Strait, Canada is crossed by a bridge of length 12,910 m, connecting the provinces of Prince Edward Island (PEI) and New Brunswick (NB). See map, Fig. 1. The structure has been named the Confederation Bridge and opened to traffic on May 31,1997. The present paper is an overview of this major project with a short discussion of its design.

In 1987 Public Works, Canada requested proposals to design, build, finance, maintain and operate for 35 years a fixed link between the provinces of PEI and NB. After six years of reviews and evaluations, the project was awarded in October 1993 to a consortium named Strait Crossing joint Venture (SCJV; see the section Acknowledgements).

The profile of the bridge is shown in Fig. 2a. The maximum roadway elevation is 59 m above Mean Sea Level (MSL), at a navigation span having vertical clearance of 49 m (Width of navigation channel is 172 m). The maximum water depth is approximately 35 m; the lowest rock level on which the footings are supported is 40 m below mean sea level.

The bridge consists of two distinctive types of bridges: (a) The approaches in which each span consists of several three metre precast segments erected by the balanced cantilever method ; (b) marine spans (Fig. 2b), in which each span (250 m) is composed of four large precast units erected by a heavy crane vessel. Table 1 gives the number of spans and their lengths in the approaches and in the marine spans. In all spans the superstructure is a single box girder of variable depths, given in Table 1 .

Figure 3a depicts the statical system for a typical three consecutive 250 m marine spans. The system consists of a continuous portal frame GCDH flanked by two spans. Each span has two intermediate hinges (A. B, E and F); an expansive joint is provided at each hinge. Thus the parts AB and EF, each of length 60 m, are simply supported on cantilevers overhanging from the adjacent portal frames.

2. DESIGN CRITERIA

2.1 Load and resistance factors

Public Works Canada set the design criteria for the bridge. These included the following requirements, which are more exigent than the demands of most bridge design codes:

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1. The bridge must have 100 year service life.
2. The load and resistance factors must be derived from full calibration process, which must be based on accepted probabilistic and reliability techniques, with safety index, β =4.0 for ultimate limit states, for 100 year life. A multi-load path structure is to be utilized.
3. The collapse of any span must not lead to the collapse of other spans. In other words, progressive collapse of the bridge must by prevented. This requirement influenced the choice of the adopted statical system for the bridge (see Section 3 below).
4. The bridge width must accommodate two lanes and wide shoulders (Total width between the guardrails is 11 m).
5. The bridge must contain, near its middle, a navigation channel of width 172 m and height 49 m.

To have a safety index of 4.00 for multi-load path components for 100 years service life, the corresponding target index for single-load-path components is found to be 4.25. The materials used, the workmanship, and the quality controls are selected to achieve the 100 year service life. The specified concrete for most parts of the structure is 55 MPa. The load factors, the resistance factors and the load combinations used in the design for ultimate and serviceability limit states are determined to achieve the specified target safety index and the 100-year service load.

2.2 Wind and current forces

The ten minute mean wind speed perpendicular to the bridge axis is considered equal to 26.5 m/s at 10 m above the mean sea level). This value is based on model tests in a wind tunnel (BLWTL, London,Ontario,Canada). The models represents the bridge in construction stages and in its completed form. The equivalent static transverse wind force on the completed superstructure carried by a typical pier in the marine spans is 3.7 MN.

The maximum current velocity is considered equal to 2.0 m/s, the maximum wave height in the middle of the Strait is assumed 4.0 m. At the pier base, the resultant wave force on a pier has horizontal, vertical(downward) and overturning moment components given below:

Depth of water m	Horizontal force MN	Vertical force MN	Overturning moment MN-m
16	8.0	4.3	53.4
25	8.1	3.5	118.5
35	8.2	2.7	192.6

2.3 Ice forces

The seasonable presence of ice floes in the Northumberland Strait lasts from December to March. Currents, waves and wind cause movement of the ice and formation of ice ridges. The average of the maximum thickness of ice floes is 2.3 m; while the maximum height of the ridges reaches 2.5 m. Extensive theoretical and experimental studies have been carried out to establish the magnitudes and time variation of the ice forces to be used in the design of piers, and to select the geometry of the ice shield shown in Figs. 4,5 and 6. At water level, the piers have a conical shape with a surface sloping 52° with the horizontal. As the ice moves against the pier, an ice floe rides up the sloping surface and fails by flexure under its own weight. With this failure type, the force on the pier is much smaller than the force which would occur if the ice is to fail, by crushing against a vertical surface. The peak value of the horizontal component of the ice force used in design is H = 16.4 MN (The factored value is 30.0MN).

Because of the inclined surface of the ice shield, the ice force has a vertical component, V = 0.14H, acting at a distance e = 4.5 m from the centre of the pier. The moment (Ve) due to this force acts in a direction opposite to the overturning moment. Thus, the conical ice shield reduces the value of H and at the same time creates the stabilizing force V and decreases the overturning moment by the value (Ve).

As the contact ice force is developed, the displacement of the pier and the deformation of the ice increase until brittle failure of the ice occurs; at which instant the structure is suddenly unloaded and vibration of the structure occurs. When the ice contact with the pier is reestablished, a new cycle of force build-up and sudden force release is started. This phenomenon of fluctuating ice force(1) observed on monitored offshore structures in the arctic, was confirmed in the experimental studies conducted for the bridge. Figure 7 depicts one of the idealized time variation of the resultant horizontal force used in the design. The abscissa of this graph represents time multiplied by the lowest natural frequency. Multiplication by the frequency is intended to select the frequency of load which coincides with the natural frequency of the pier to obtain the maximum dynamic response of the structure. The graph in Fig. 7 is used in design for all the piers, which because of difference in height have different natural frequencies.

It is noted that the peak value H = 16.4 MN is not reached in Fig. 7. This is so because the peak force occurs randomly at irregular time intervals; thus the peak value 16.4 MN need not be considered in dynamic analysis; but must be considered in static analysis. However, from numerous dynamic analyses, it is concluded that with the amplification factor caused by the dynamic effect of fluctuation of the ice forces, the design is governed by static analysis using the peak value, H = 16.4 MN.

3 . CONSIDERATION OF PROGRESSIVE COLLAPSE

This section discusses the contract requirement: "... the bridge shall be designed at a level of risk where the failure or collapse of any one span or section does not lead to progressive failure or collapse of other spans or section" . Thus, for example, with the loss of support A of the drop-in unit AB in Fig. 3a, failure must be limited to one span. The loss of support at A can be due to an accidental collapse of the cantilever by ship collision or by exceptional ice force on a pier. At one stage of design, it was proposed to replace the hinges at B and F by casting joint; thus making the statical system as shown in Fig. 3b. The purpose was to eliminate at B and F the need for expansion joints, support bearings, corbells and dapped ends of the drop-in units and the cantilever tips. Does this proposal affect the vulnerability of the system to progressive collapse?

Accidental loss of the support at A of the drop-in unit AB in Fig. 3a will cause it to fall down and separate from the remainder of the structure. On the other hand, the same accident in the system in Fig. 3b, will produce flexural failure and formation of a plastic hinge at a weak section, in the vicinity of B (location of the eliminated hinge, Fig. 2a). Figure 8 represents the falling drop-in unit AB, with end A moving in a circular path, while end B continues to be attached to the part of the structure situated on the right-hand side of B. Dynamic analyses and experimental studies(2) are conducted to study the variation of the internal forces, X and Y with time during the motion of AB .

Figure 9 (from Ref. 2) shows the variation of the internal forces X and Y with time before the end A of the falling drop-in unit hits water, ice or sea floor. In the analyses; a spring, a damper or a combination of the two is introduced at end A to calculate the reaction of the surface which the end hits. A wide range of values of the spring stiffness or the damper constant is employed in the analyses. The results presented in Fig. 9 are for the case of fall on a spring. The figure shows that at the end of the fall in the air, Y = 1.26 W and X = 1.12 W; where W is the weight of the drop-in unit. These values are, respectively, 2.5 and 2.2 times the value of Y in the static position before the fall. After the end A of the falling part hits the spring, much higher values of X and Y are developed. Similar results are obtained when the spring is replaced by a damper. It would be very costly to design the frames with sufficient strength to resist these forces. For this reason, the proposal of eliminating the hinges at B and F (as done in Fig. 2b) is abandoned to retain in the final design the statical system in Fig. 3a. At the time when Fig. 9 is prepared the length of the drop-in unit is assumed 80 m; subsequently this length is reduced to 60 m.

4. THE APPROACH SPANS

The heavy-1ift crane on the floating vessel used for installation of large units in the marine spans cannot be employed in the approach spans because of shallow waters. For this reason, the conventional precast segmental balanced cantilever method is used to construct the approach spans (each of length 93 m). Figure 10 shows a typical balanced cantilever and a pier. Hollow rectangular precast segments of height 3.6 and 5.2 m, assembled by post-tensioned cables are used to build the pier. The balanced cantilevers consist of 30 precast segments assembled by post-tensioning and installed from a launching truss. Expansion joints are provided at midspan. Steel beams inside the joints allow relative movement of the cantilever ends only in the longitudinal direction; but the steel beams restrain the relative translation in the transverse directions as well as the relative rotations.

5. THE MARINE SPANS

This section describes the construction of 45 marine spans; of which 43 spans have a typical length of 250 m. Each span is divided into four large units fabricated in a precasting yard and installed in their final position in the Strait with the use of a powerful floating crane (called the Svanen) Restricting the number of units to this small number is intended to minimize the offshore operations required to connect the units with cast-in-situ concrete or with prestressing tendons.

The four units are referred to as (Fig. 4) the pier base, the pier shaft, the main girder and the hinged drop-in or the fixed drop-in. Each of these units is described below:

5.1 The Pier Base

This unit is the part of the pier from the sea bed to 4.0 m above mean sea level. The unit is fabricated in four segments separated by casting joints (Fig. 11): footing ring, conical footing, cylindrical part of footing with varying depth and an upper part of footing (Fig. 6). The footing ring has either a 22 m or 28 m outer diameter. The larger diameter is used for eight piers located at a water depth greater than 27 m. With each of these footing diameters, the dimensions of the conical footing are kept constant. The variation of water depths at pier locations is accommodated by varying the height of the cylindrical part of the pier base unit. The upper part of the unit is a stump to receive the ice shield. A11 piers have identical stumps. The pier base unit, weighing 3,500 to 5,200 ton, is transported to the pier location in the Strait and seated on three prelevelled footing supports. These are precast concrete pieces placed in a ring-shaped excavation in the sea floor. The space below the ring-shaped footing and the bottom of the excavation is filled with underwater concreting.

5.2 The pier shaft

The pier shaft unit consists of the ice shield and the pier shaft (Fig. 11). The ice shield, is a truncated hollow cone, fitting the stump of the pier base unit. The height of the cone extends from 4.0 m below to 5.00 m above the mean sea level (Fig. 6). The outer diameter of the cone is 20 m at the bottom. The generatrix of the cone makes an angle 52° with the horizontal. The ice shield is made of concrete with specified strength 55 MPa, with the exception of the middle 2.0 m of the height, which is made of high performance concrete with specified strength 100 MPa. The pier shaft varies in cross section from rectangular hollow section of outer dimensions 5 x 10 m at the top to a hollow octogonal section of outer dimension 8.0 m at the bottom. The shaft has an extension, at its lower end, with constant hollow octagonal section for the piers with variable height in the vicinity of the navigation channel. The wall thickness in the pier shaft is 600 mm. The individual weight of the majority of the pier shait units, is 4,000 ton. Post-tensioned tendons and grout connects the pier shaft unit and the pier base unit.

5.3 The main girder and the drop-in units

The main girder unit is a box girder of length 192.5 m and varying depth from approximately 5 m at the ends to 14.0 m at the pier face. The main girder unit, weighing 7,800 ton, is placed above the pier shaft unit and connected to it by epoxy and post-tensioned tendons, forming cantilevers of lengths from the pier centre lines to the tips equal to 95 m and 97.5 m (Fig. 4). A concrete piece, match cast against the bottom of the main girder, at the location of the pier shaft, serves as template to facilitate accurate positioning of the main girder unit (Figs. 6 and 11).

The main girder unit consists of seventeen segments fabricated in the precasting yard by the balanced cantilever method (Fig. 11). The pier segment, of length 17 m, is reinforced by a structural steel diaphragm in the shape of an inverted letter V to help in the transfer of unbalanced moment between the main girder and the pier shaft. The remaining sixteen segments have lengths varying between 7.5 m and 14.5 m, but approximately the same volume. The cantilever whose length is 95 m has a tip in the shape of a corbel to receive the hinged drop-in girder. The tip of the cantilever at the other end is a casting joint to receive cast-in-situ concrete and post-tensioned tendons to connect the main girder to the fixed drop-in unit.

The fixed drop-in unit, having a length of 54 m, connects the tips of two cantilevers with a 0.5 m closure cast-in-situ joint to form a portal frame with two overhangs. The weight of this unit is 1,200 m. The hinged drop-in girder, of length 60 m, has dapped ends to accommodate simple support bearings on the corbels. The weight of the hinged drop-in unit is 1,600 ton. The depths of the drop-in units and the main girders are varying such that the soffit has the appearance of a continuous parabolic surface, with girder depth 4.5 m at midspans and 14 m at the piers (Fig. 2a).

6. CONSTRUCTION PROCEDURES

The bridge units are transported within the precasting yard, and out to a jetty, using Huisian Sliding System (made by Huisian Iterc, Holland, Fig. 12). These sliders are large low-profile hydraulic machines capable of lifting and transporting large and heavy units. The machines are provide with teflon plates and stainless steel belt to slide on concrete tracks covered with stainless steel.

A heavy lifting crane mounted on a catamaran vessel, called the Svanen, is used to lift all bridge units off the jetty, transport them out in the Strait and install them in their final positions (Figs. 13 to 16). Description ofthe precasting and the production procedure of the units are presented in the remainder of this section. The marine operations required to install the units in the Strait are described in the following section.

The four bridge units of each of the marine spans were fabricated in a precasting yard at the PEI end of the bridge. The top soil was removed from a 60 hectare area and the ground surface was levelled at constant elevation of 5.0 m above mean sea level. Movement of the Huisian Sliding System dictated that all elements must be cast 5 m above ground level. This allowed the sliders to move underneath the precast units and lift them from their supports and transport them to a new position on a casting line. Thus, the system dictated that the positions of production and movement of the units must be aligned over straight lines and preset tracks. Figures 17 and 20 are pictures of the precasting yard and a pictorial view of the same in a particular week (144 weeks after the beginning of the project (October 1993)).

Three casting/storage lines are used for the pier bases; two of which are exclusively for the pier having 22 m diameter ring footing; the third is for the piers with footing diameters 22 and 28m. The ringfooting and the conical footing are cast in the first position of each line. The sliders then move the cast piece to a storage area, where the barrel and the stump are completed.

A single line is used to cast the pier shafts. The ice shield is cast, up to a height of 10 m, in the first position of the line and in a nearby position; then the cast piece is moved to a storage spot on the line, where the pier shaft is completed in multiple lifts (Fig. 20).

The segmental casting of the main girder units is done in one line, with the units changing location while the forms for the segments are stationary. This is contrary to the conventional balanced cantilever segmental construction routine, where the girder is stationary while the forms move outwards. The fabrication of the main girder unit starts by producing the match cast template (Fig. 6). The template is used as a form for the soffit of the hammerhead (central segment) of the main girder. After casting the hammerhead, its template is moved to a storage location, and the process of balanced cantilever starts. Every week a segment on each side of the cantilever is cast and prestressed, and the completed part of the unit advanced to a new location on the casting line. This routine produces a new main girder unit every week; within each unit, the ages of the concrete of individual segment is one week older or younger than the adjacent segment.

Both the fixed drop-in and the hinged drop-in units are cast in single pours in the same form. The cast units are prestressed temporarily at the top, so they can be transported and stored as balanced cantilevers.

The fixed drop-in and the hinged drop-in units are cast in single pours in the same forms, with lengths 48 m and 52 m, respectively. The units are provided with temporary prestressing so that they can be transported and stored as double cantilevers. In the storage location, a 3 m segment is added at each end of the fixed drop-in units; while a 4 m dapped shape corbels are added at the ends of the hinged drop-in units (Fig. 4).

7. MARINE OPERATIONS

The following marine operations are involved to complete the marine spans:

Step 1: At the pier locations, carry out geotechnical investigations to determine the depth of the overburden, the strength of the subgrade and the appropriate foundation elevation.

Step 2: Use dredging equipments to remove the overburden to a level 2.0 m above the foundation elevation determined in Step 1 .

Step 3: Centre a steel template on the area dredged in Step 2 to act as a guide marking the inner edge of a ring-shaped trench 6 m wide to be dredged down to the foundation elevation. This trench is to receive the ring footing of the pier.

Step 4: Clean the trench to install three precast concrete pads, each 5 m in diameter, with their x,y and z location controlled to a specified tolerance. These become temporary footing supports for the pier bases.

Step 5: Bring a pier base unit by the Svanen, and place it on the three concrete footing supports installed in the preceding step.

Step 6: After placement of the pier base unit and checking the accuracy of its position within specified tolerance, place by means of a jack-up barge, 55 m^3 (approximately) of tremic concrete under the ring footing to ensure uniform bearing capacity.

Step 7: When the tremic concrete reaches a specified strength, use the Svanen to place a pier shaft unit on the stump of the pier base (Fig. 11). Employ jacks placed on the top of the stump to adjust the position of the pier shaft unit in the correct position. Seal the space between the stump and the pier shaft unit to fill in the space with grout. When the grout reaches specified strength, secure the continuity of the pier base unit with the pier shaft unit by prestressing tendons.

Step 8: Place the concrete template piece on top of the pier shaft to serve as guide for accurate placement of the main girder unit. Adjust the template position to more strict tolerance, then grout the space between the template and the top of the pier shaft. After grout hardening, attach the template to the pier shaft temporarily by Dywidag prestressed bars.

Step 9: Use the Svanen to carry a main girder unit above the template coated with epoxy and fit the underside of the main girder unit to the match-cast shear keys provided in the template. Use horizontal guides and vertical jacks, installed on top of the pier shaft, to dampen motion of the main girder unit. At the termination of this step a T-shape structure is completed (Fig. 21).

Step 10: Repeat Steps 1 to 9 at the adjacent pier to produce another T-shape structure consisting of pier base unit, pier shaft unit and main girder unit.

Step 11: This step is to install a fixed drop-in unit between two completed T-shape structures. Use the Svanen to carry a drop-in unit, provided with protruding steel supports at the ends, in the appropriate position above the tips of the cantilevered girders. Gradually release the weight of the drop-in unit and use horizontal and vertical jacks to control the relative movement of the unit and the tips of the cantilevered main girders. As the weight is released, the two cantilever tips tend to move towards each other, activate the horizontal jacks to restrain this movement, thus providing compressive forces in the drop-in unit and in the cantilevers. Pour concrete to fill in the gaps between the drop-in unit and the adjacent main girder units, and secure the continuity of the three units by prestressed tendons. At the end of this step a portal frame with two overhangs is completed.

Step 12: Repeat Steps 1 to 11 to produce other frames. Install the hinged drop-in units to complete the marine spans (Fig. 22).

While the marine operations are done, the finishing works on the bridge progressed. These included installing expansion joints, safety barriers, lighting, traffic control systems and signs.

8. SUMMARY AND CONCLUSION

The paper presents a concrete bridge crossing a strait of width thirteen kilometres. The design criteria and the method of construction are described. The project is completed within 44 months after awarding the contract. The marine operations have been reduced to a minimum by the extensive use of precasting and careful planning of the precasting yard and the equipments used for transporting and lifting large prefabricated units.

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2. Ghali, Amin and Tadros, Gamil, "Bridge Progressive Collapse Vulnerability", Proceedings ASCE, Structural Journal, Vol. 123, No. 2, February 1997, pp. 227- 231.

3. Tadros, Gamil, "The Confederation Bridge, Canada: An Overview", Canadian Journal for Civil Engineering, Vol. 24, No. 6, December 1997 (in print).

ACKNOWLEDGEMENTS

The conceptional and the preliminary designs of the winning bridge proposal were done by Strait Crossing Inc. (SCI, 1987-1993). In October 1993 the contract was awarded to a consortium named Strait Crossing Joint Venture (SCJV), consisting of Strait Crossing Inc., Canada, Dumez-GTM, France, Ballast Nedam, the Netherland and Morrison Knudsen, U.S. (who opted out during construction). A joint venture of J. Muller International, San Diego, USA and SLG Stanley, Calgary, Canada did the detailed design, which was reviewed by Buckland and Taylor Ltd., Vancouver Canada as independent engineer.

The first author of the present paper was leader of the group of engineers for the conceptional and preliminary designs for SCDI; after 1993, he was chairman of Technical Review Committee of SCJV. Other consulting engineers (including the second author of the present paper) and subcontractors who worked on the project on behalf of SCJV are listed in Ref. 3.

Table 1 Span lengths and Girder depths

Part of bridge	Length m	Number of spans	Span lengths m	Girder depths at mid-span m	Girder depths at pier m	Construction method
PEI approach	555	7	30, 60, 5 @ 93	3.0	5.1	Segmental balanced cantilever
Marine spans	11 080	45	165, 43 @ 250, 165	4.5	14.0	Large precast units
NB approach	1 275	14	13 @ 93, 66	3.0	5.1	Segmental balanced cantilever

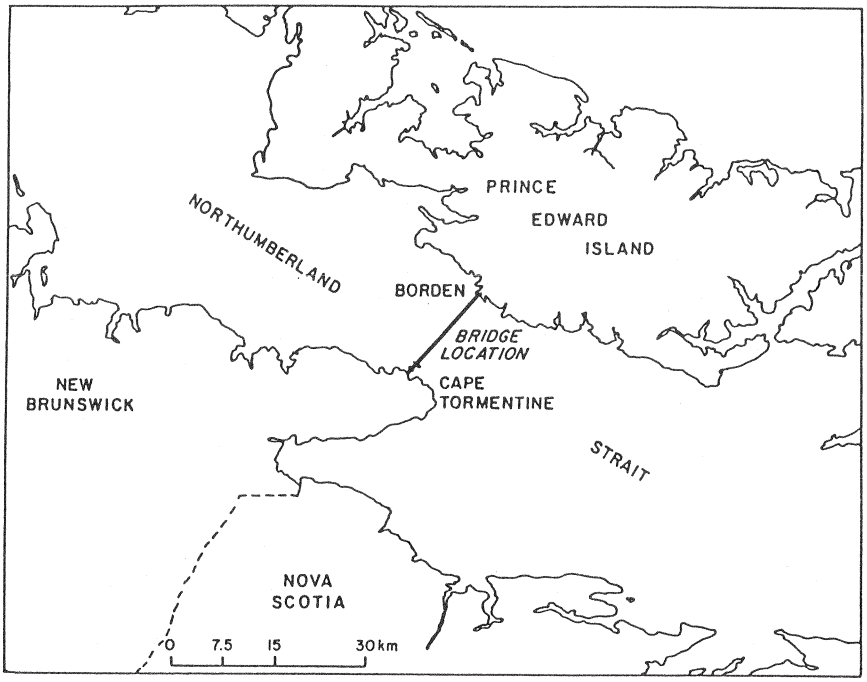
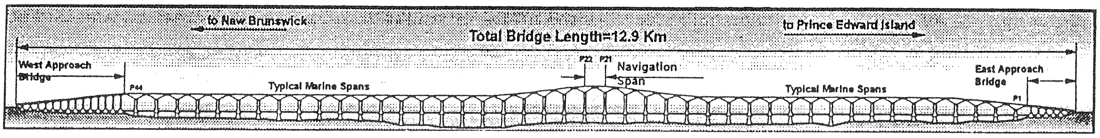
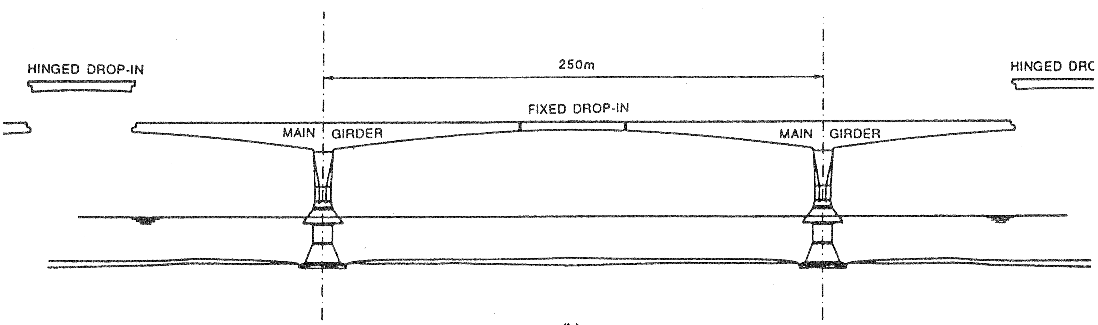


Fig. 1- Map at bridge location



(a)



(b)

Fig. 2- Elevation of the Confederation Bridge: (a) bridge profile; (b) detail of marine spans

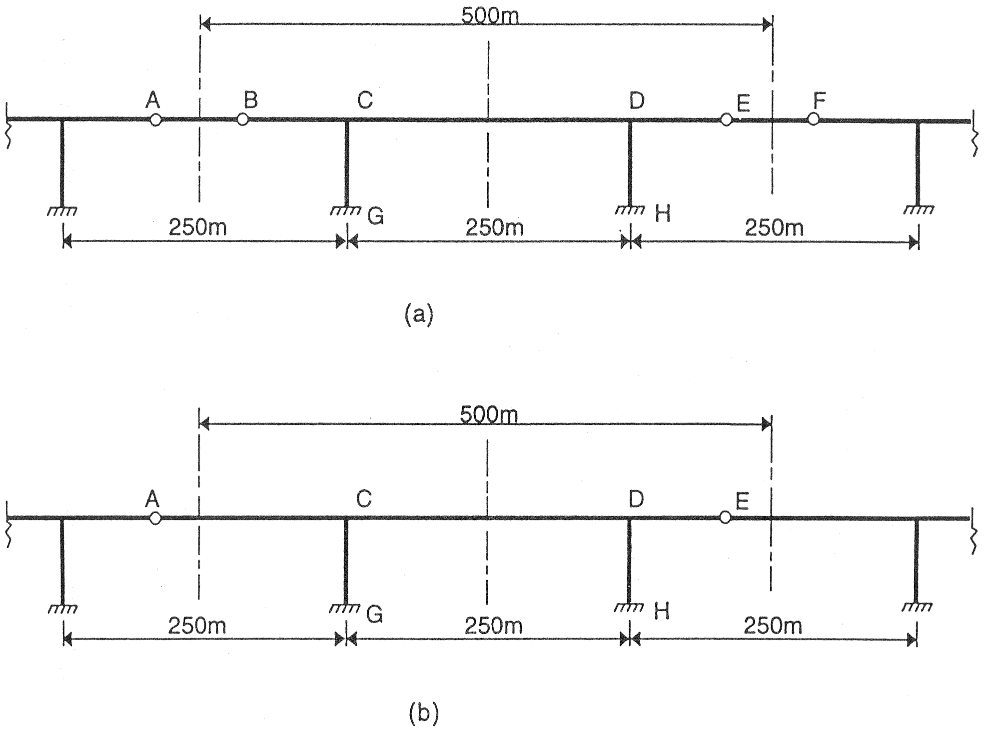


Fig. 3- Two statical systems for the marine spans: (a) system adopted in final design; (b) a proposal abandoned in consideration of its vulnerability to progressive collapse

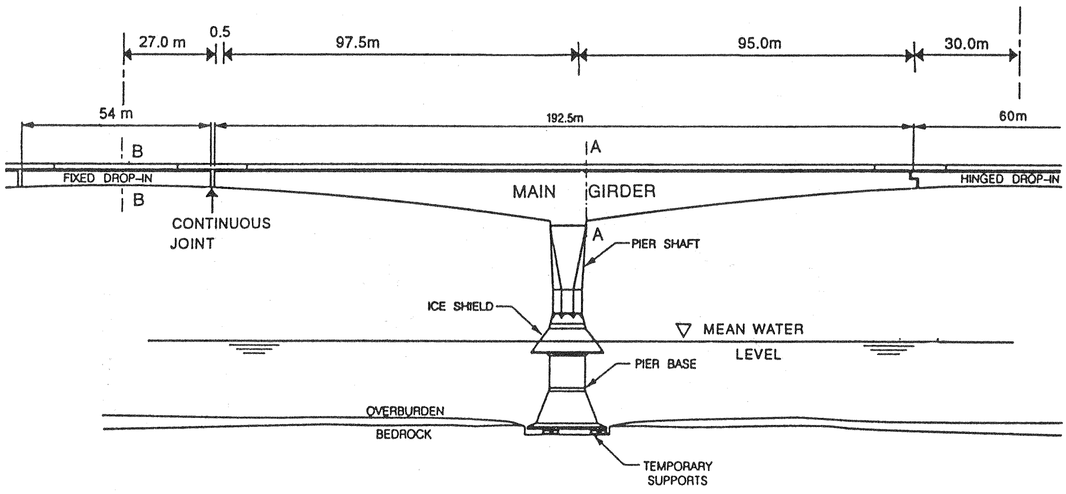


Fig. 4- Elevation of a typical pier and two halves of marine spans

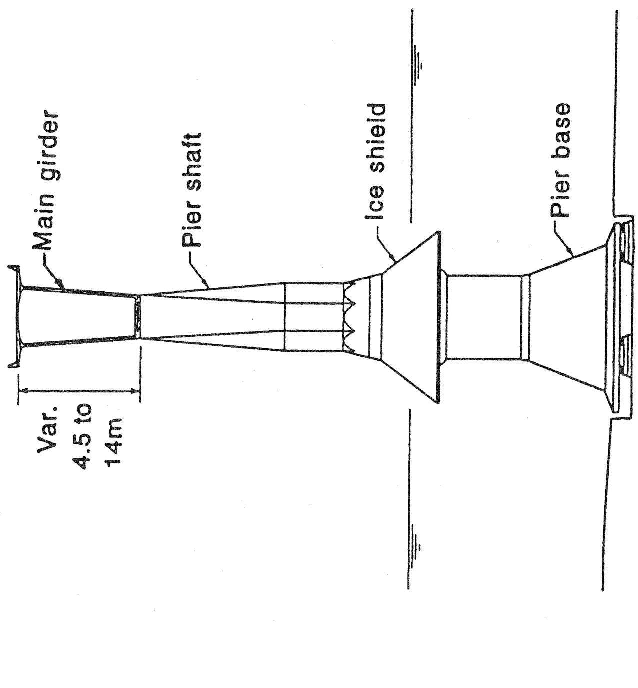


Fig. 5- Cross section in a marine span near a pier

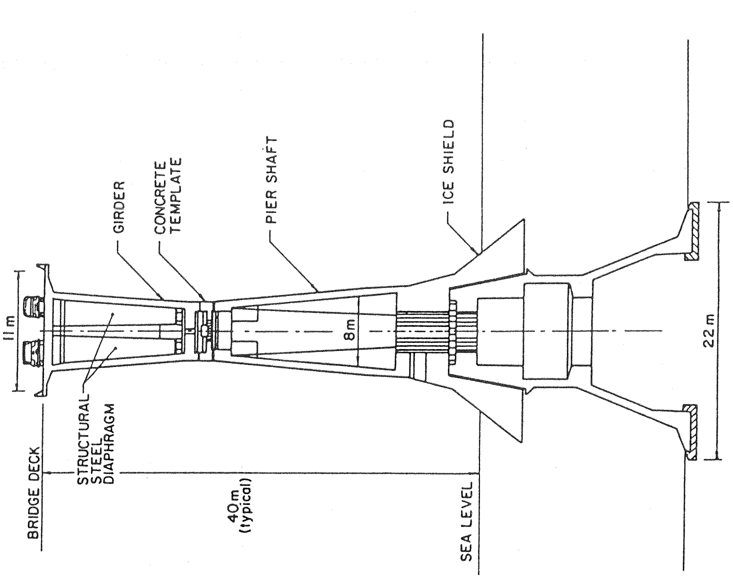


Fig. 6- Cross section through pier centre line in the marine spans

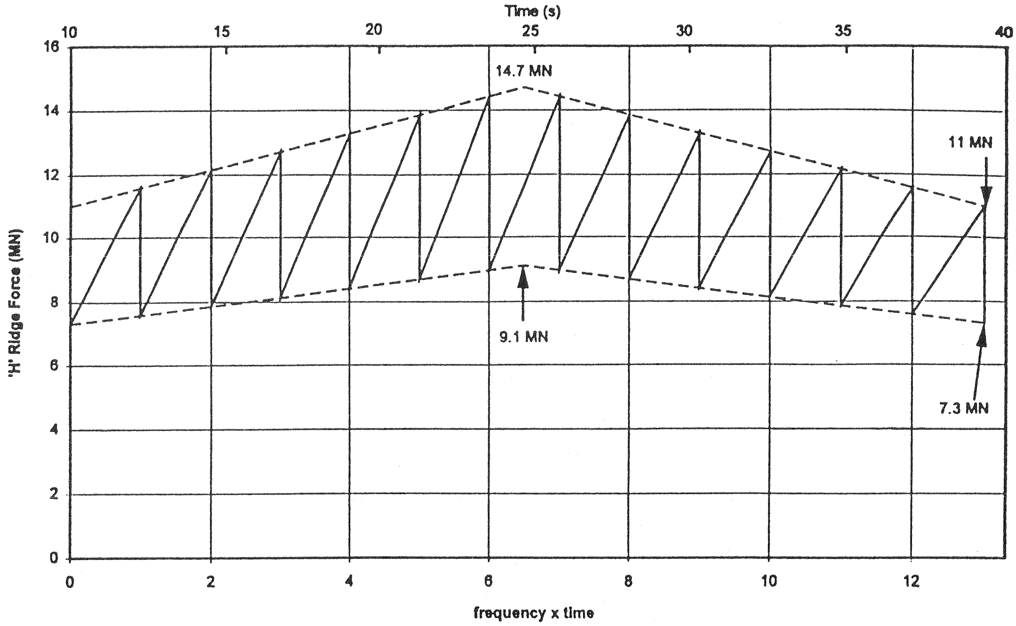


Fig. 7- Idealization of the variation of the horizontal component, H of ice force on a pier in the marine spans. The scale on top edge of the figure is used for a pier whose lowest natural frequency = 0.44 Hz. (Note the initial starting time is selected at 10 s.)

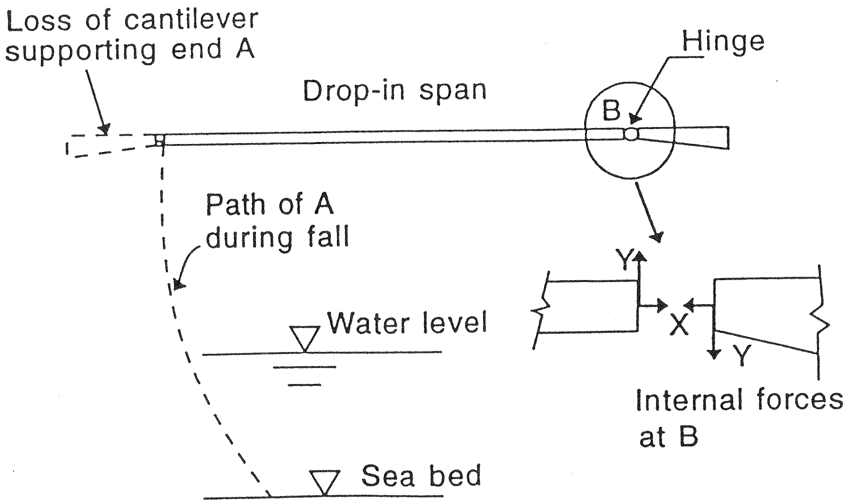


Fig. 8- Definition of the internal forces at a plastic hinge developed at B in an accident in which the support at A of the drop-in unit is lost in the structural system shown in Fig. 3b.

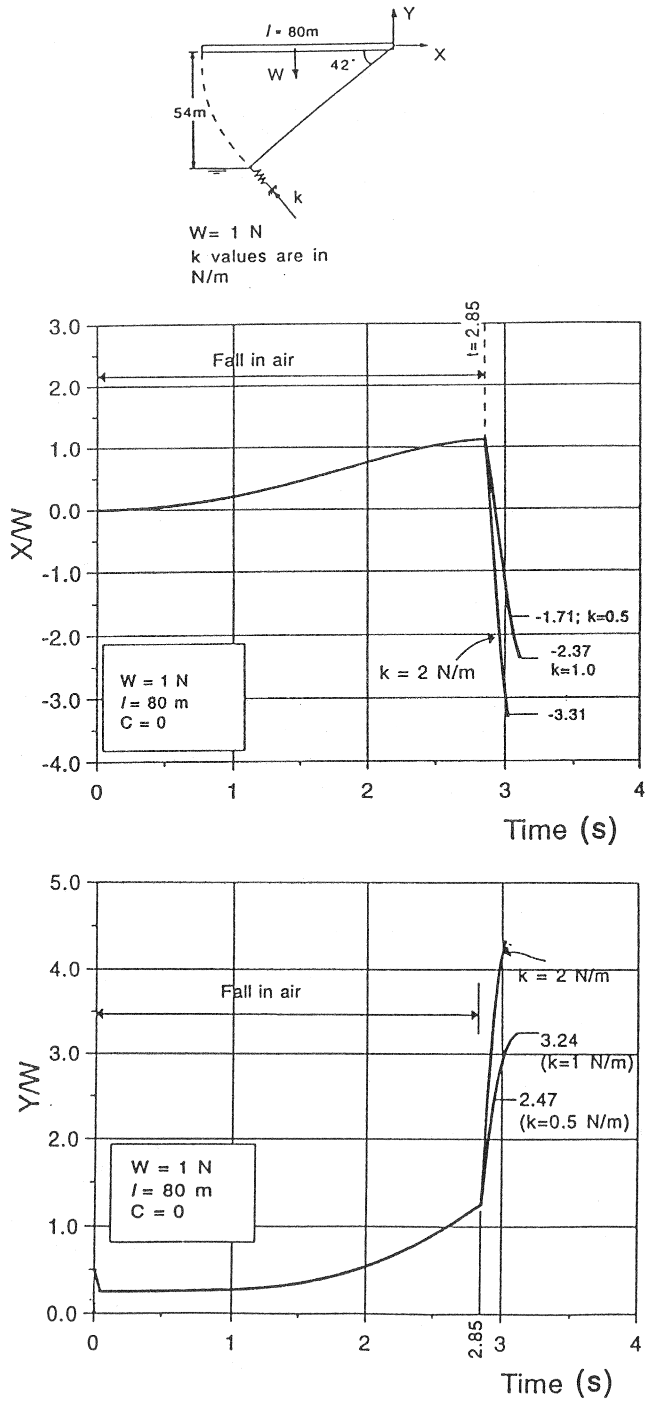


Fig. 9- Variation of the forces X and Y at the right-hand hinged end of a beam, when the left-hand end is allowed to fall down on a spring with varying stiffness.

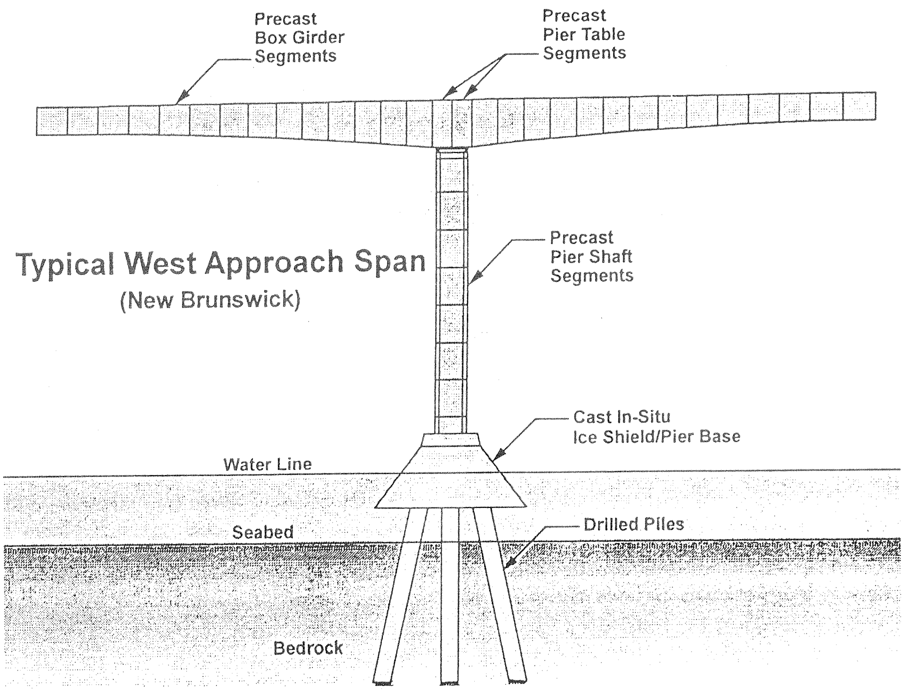


Fig. 10- A pier and a balanced cantilever built in segments. The part shown represents typical part between centre lines of adjacent spans, each of length 93 m of the bridge approaches

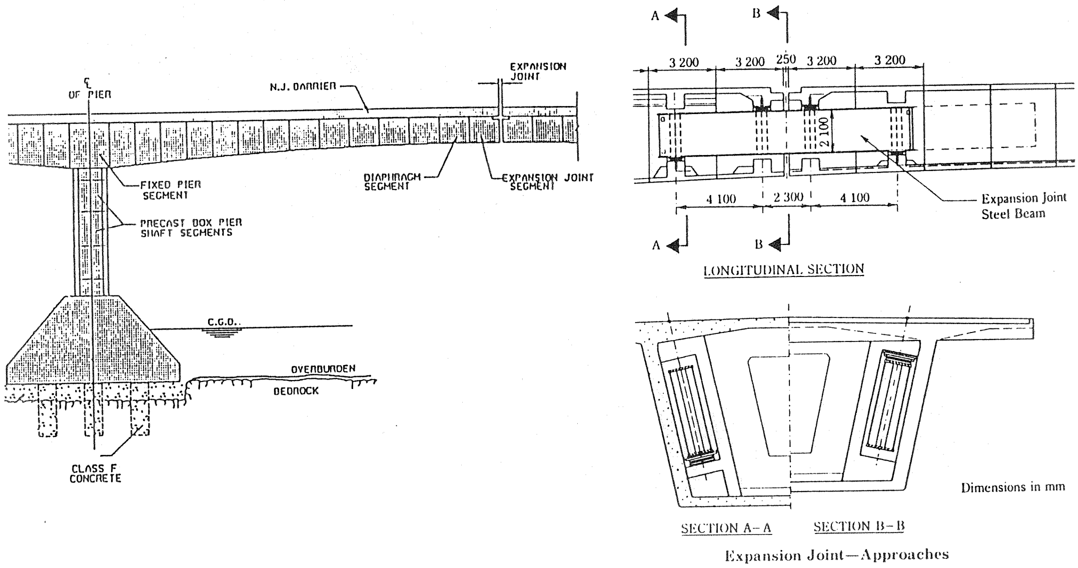


Fig. 10*- Expansion Joint - approaches

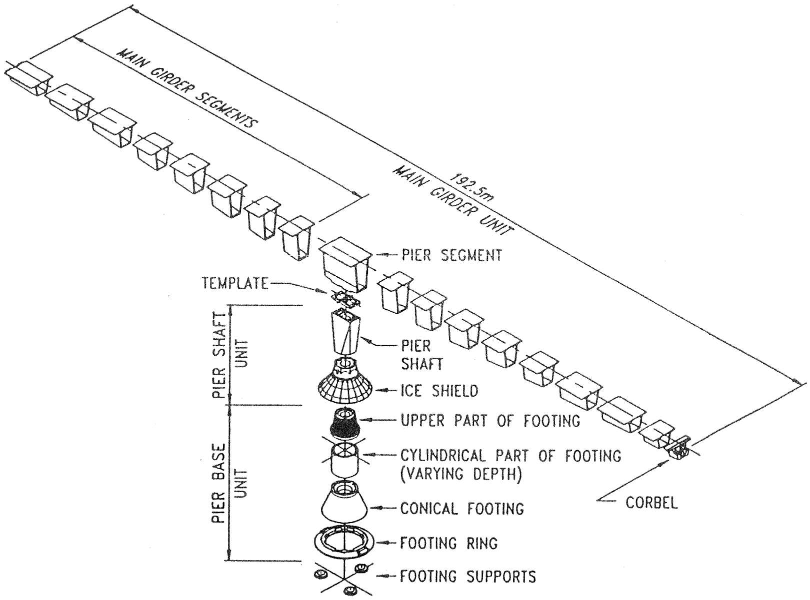


Fig. 11- Segmental components of the prefabricated units

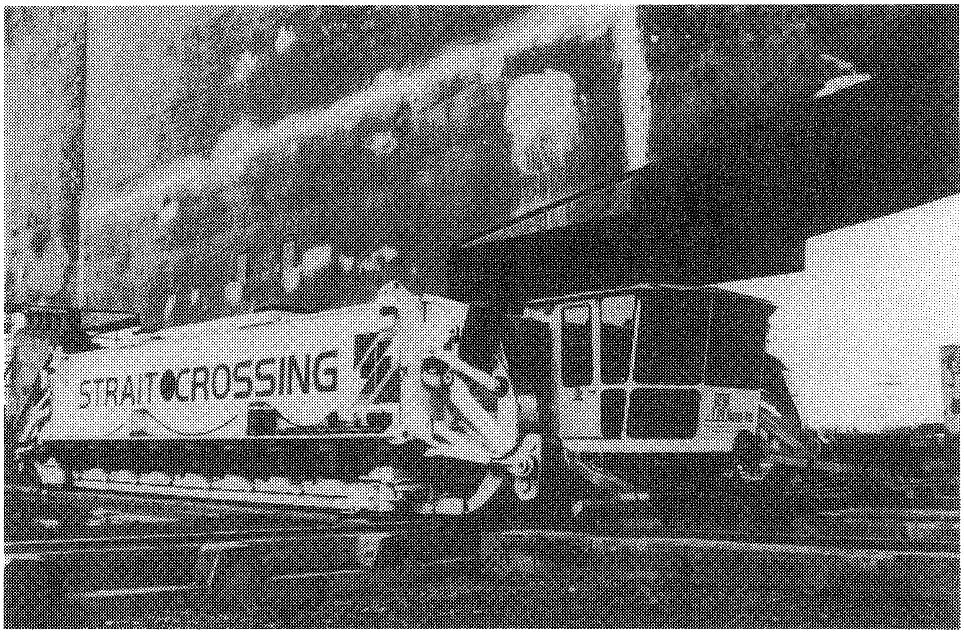


Fig. 12- Huisian Sliding System carrying a main girder unit

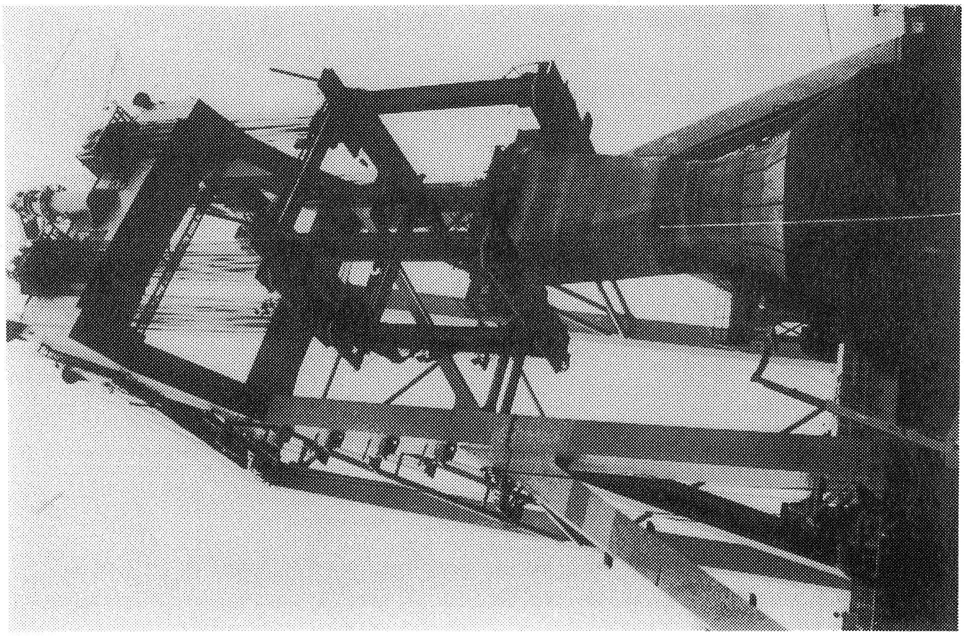


Fig. 14- The Svanen lifting a pier shaft unit

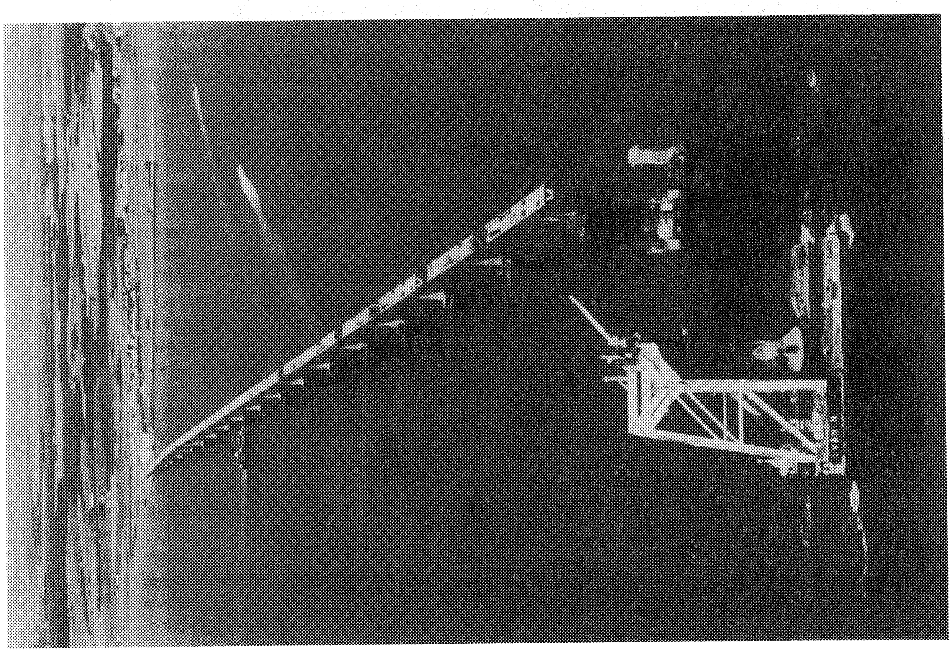


Fig. 13- The Svanen lifting a pier base unit

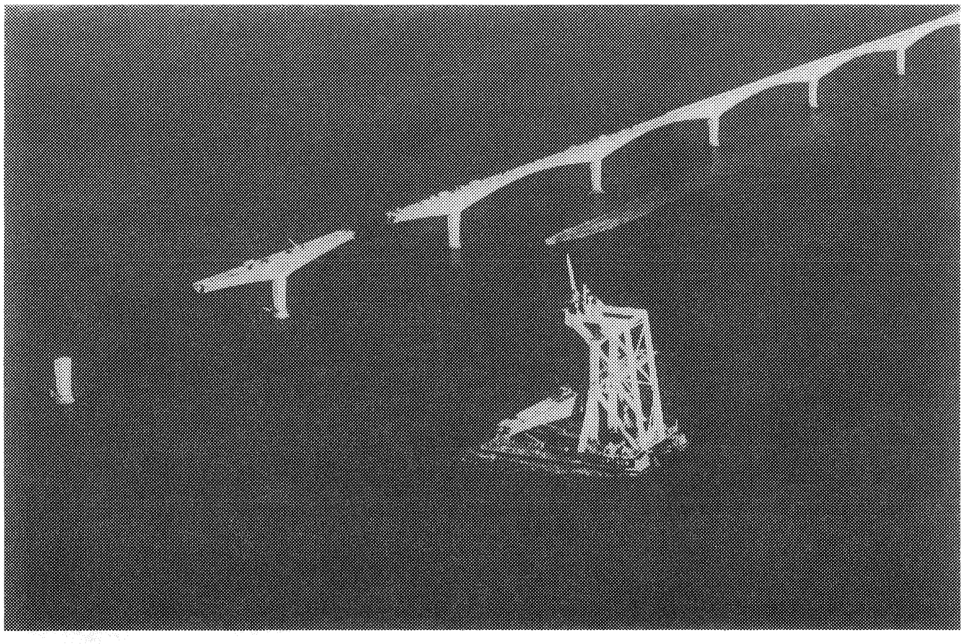


Fig. 15- The Svanen lifting a main girder unit

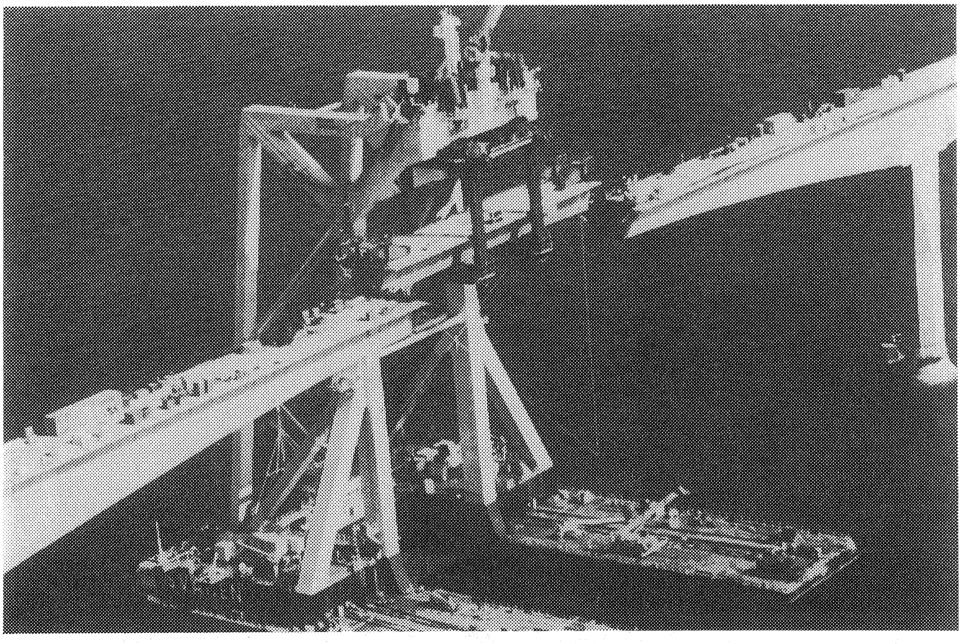


Fig. 16- The Svanen lifting a drop-in unit

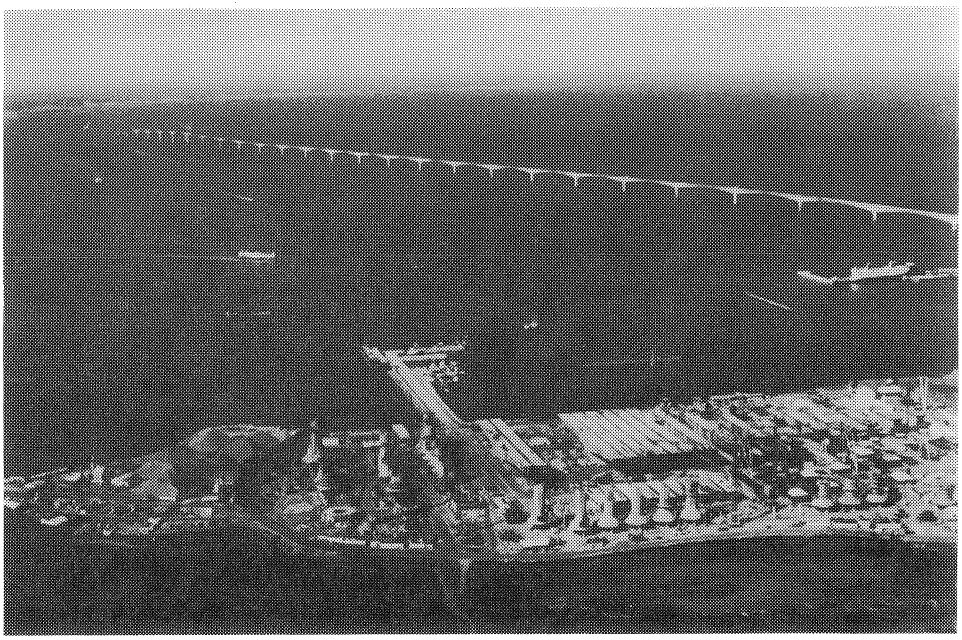


Fig. 17- View of bridge with precasting yard

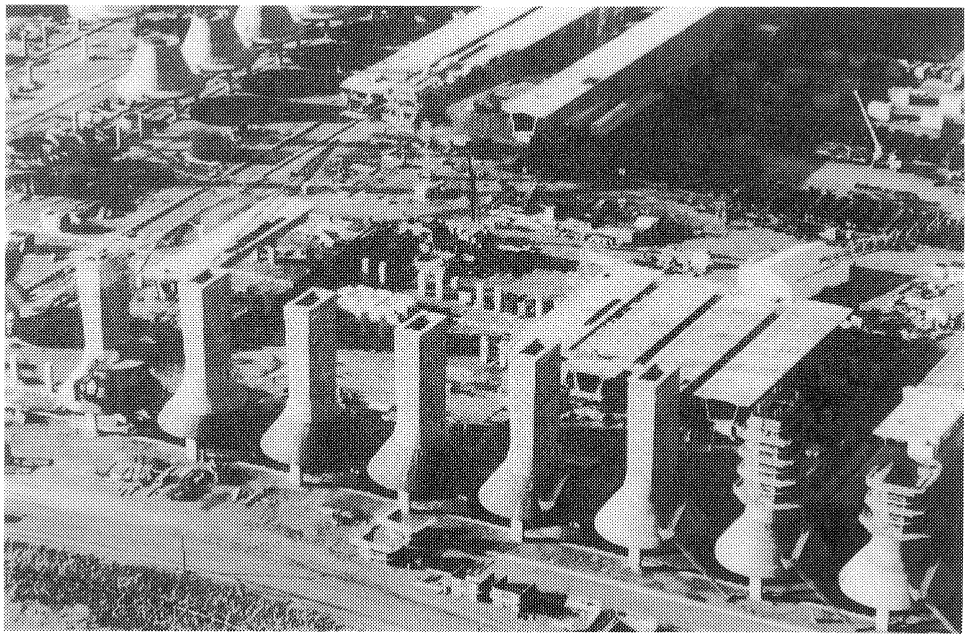


Fig. 18- View of precasting yard with pier shaft units in the forefront

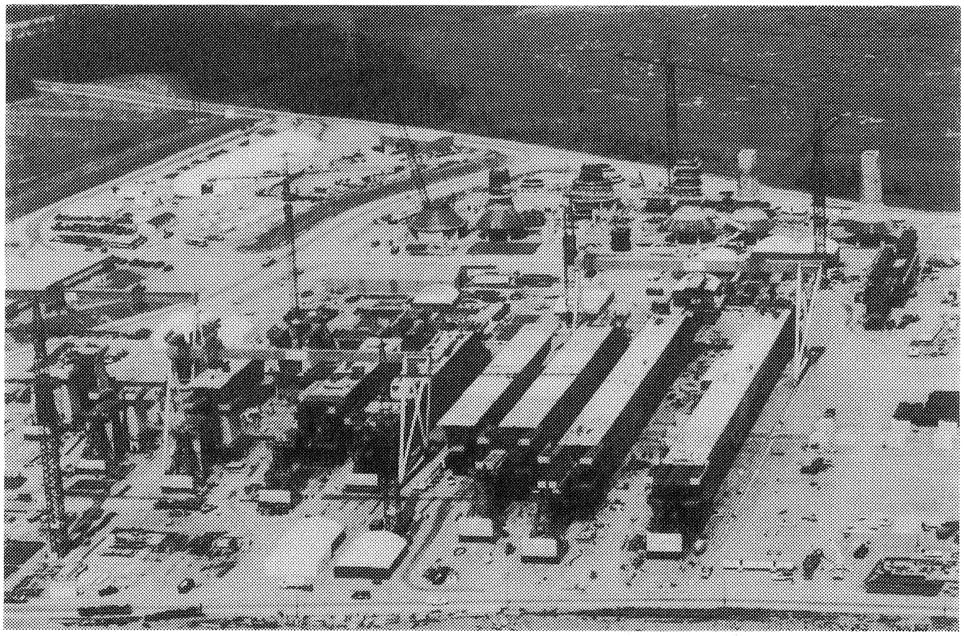


Fig. 19- View of precasting yard with main girder units in the forefront

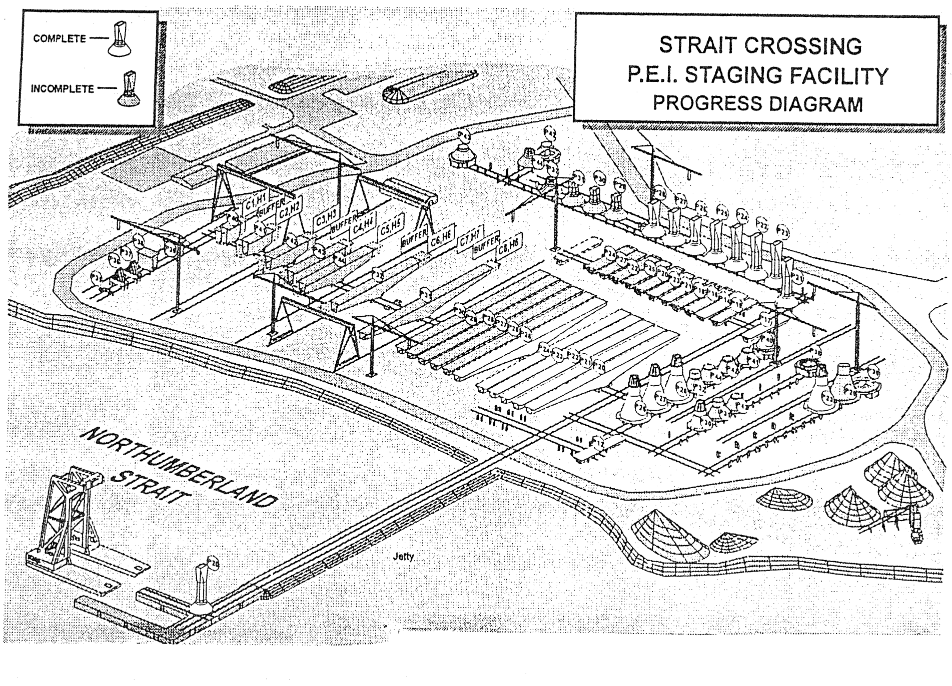


Fig. 20- Pictorial view of the precasting yard at 144 weeks after the beginning of the project (October 1993)

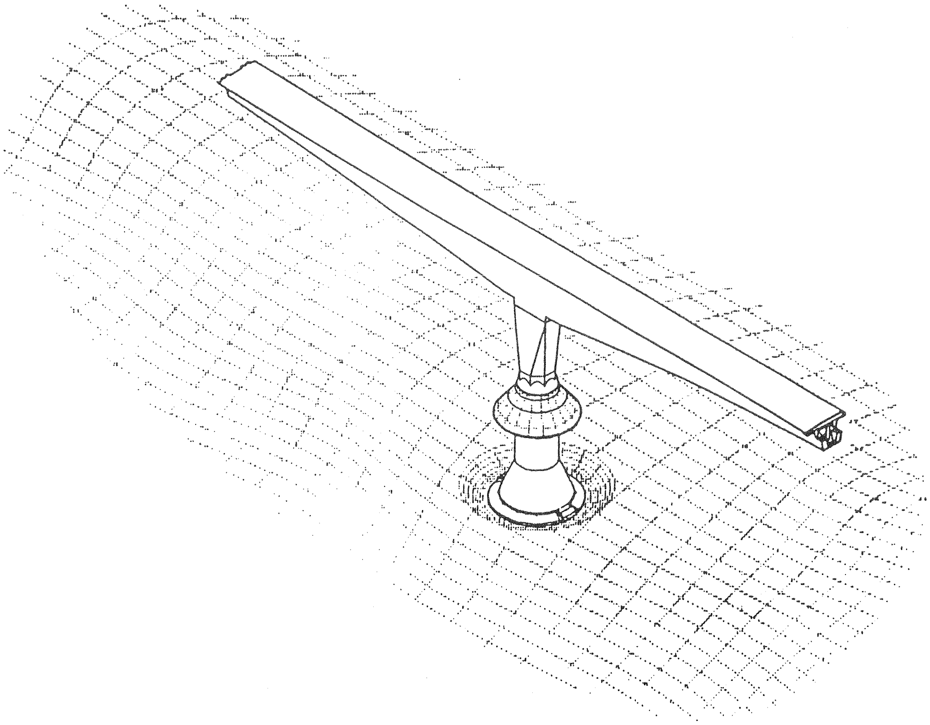


Fig. 21- A pier with two overhangs ready to receive the drop-in units

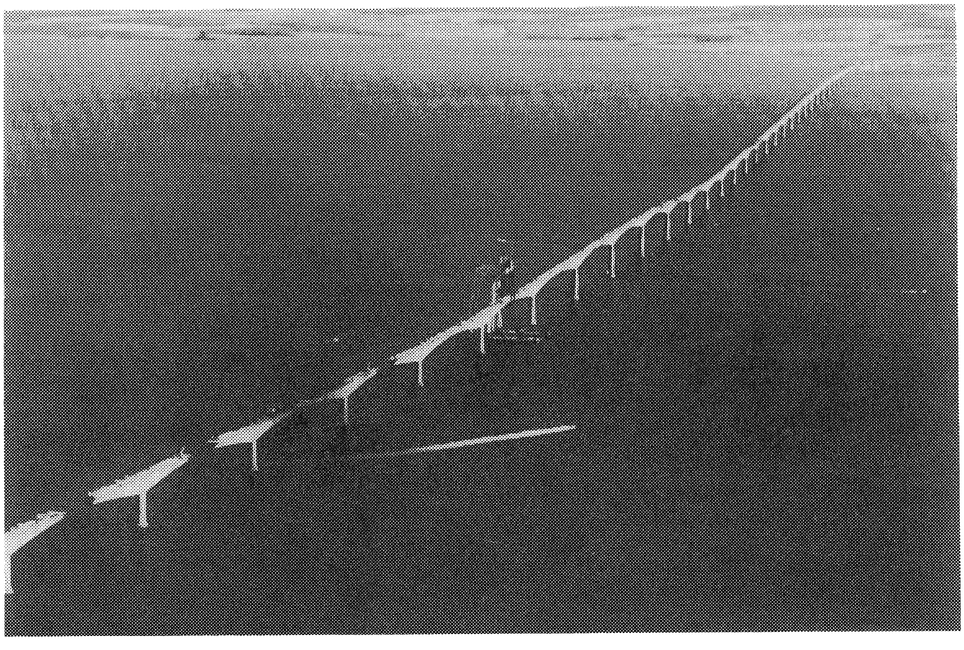


Fig. 22- View of the bridge near the end of marine operations