Cable-Stayed Houston Ship Channel Crossing

HOLGER S. SVENSSON AND THOMAS G. LOVETT

The bridge crosses the Houston Ship Channel 20 miles east of Houston between the cities of Baytown and LaPorte, Texas. It is the first dual superstructure cable-stayed bridge, and its total deck area of about 350 000 ft^2 makes it one of the largest cable-stayed bridges to date.
Each of the dual cable-stayed bridges has a composite superstructure girder with a main span of 1250 ft and a navigational clearance of 175 ft.
The twin concrete towers with a double diamond shape configuration rise 426 ft above the ground. In the transverse direction they carry the loads by truss action, in the longitudinal direction they are fixed to the foundations and elastically supported by the stay cables.

Each of the two girders is 78'-2" wide and consists of a 5'-3" deep steel grid. Exterior main edge girders and transverse floor beams at 17 ft spacing are plate girders with an 8 in composite concrete roadway slab. An additional wearing surface is 4 in thick. The roadway slab was designed for composite action under dead and live load.

The stay cables vary between 19 and 61 strands of 0.6" diameter. Their corrosion protection consists of Polyethylene (PE) pipes with cement grout. In addition, they are wrapped with a white weather-resistant tape.

Construction started in 1987, completion of the bridge is anticipated in 1992.

1 INTRODUCTION

The Houston Ship Channel Crossing, located 20 miles east of Houston, Texas, will replace the existing two lane Baytown Tunnel with a dual eight lane high level structure connecting Baytown and LaPorte. Presently, trucks with hazardous cargoes are prohibited from using the tunnel and must make a 16 miles detour. Completion of the bridge will allow the tunnel to be closed and permit the deepening of the ship channel.

At the bridge site the water is about 1500 ft wide with a required navigation clearance of 175 ft over a width of 600 ft. One tower is located on the existing levee, the other is placed in shallow water. For protection against ship collision the second tower is surrounded by an artificial island which also serves as a staging area for the construction.

The approach bridges were designed by the Texas Department of Highways and Public Transportation, using conventional precast prestressed girders with spans of up to 143 ft.

In accordance with Federal Highway Administration requirements alternate steel and concrete designs were prepared. In 1987 four bids ranging from 91.3 to 126.5 million dollars were received for the total project including the approach bridges, with all bidders selecting the subject steel-composite main bridge.

The structural design was done in accordance with the AASHTO Bridge Specification, amended as appropriate by other US and international codes.

All concrete and steel members were sized by the load-factor method and checked under working loads for fatigue and deflections.

Concrete for the roadway has a 7,000 psi compressive strength, concrete for the towers has a 6,000 psi compressive strength. All reinforcement is Grade 60, the structural steel is A 572, Grade 50.

2 GENERAL LAYOUT

The cable-stayed bridge is continuous over its length of 2214 ft, see Fig. 1. The girder is supported by stay cables in a semi-fan arrangement with anchorages at deck level at about 51 ft intervals.

The support conditions render a completely symmetric structure. The girders are connected to each tower with flexible neoprene bearings, 10 in high, which allow temperature movements by shear deformation and distribute longitudinal forces equally to both towers. The anchor piers are connected to the girders by rotational bearings. They are slender enough to allow temperature movements of the girder by deflection. The beam rotations are taken by strip seals above the anchor piers. All expansion movements take place at the two 30 in modular expansion joints on top of the piers at the ends of the two 130 ft flanking spans. Transverse wind loads are taken by bumpers at the towers, and at the anchor piers.

3 AERODYNAMIC INVESTIGATION

The bridge is located in a hurricane prone area near the Gulf of Mexico. The basic design wind speed for a 100 year

H.S. Svensson, P.E., Manager North American Projects, Leonhardt, Andrä u. Partner, Lenzhalde 16, Stuttgart, Germany
T.G. Lovett, P.E., Associate Vice President, Greiner Inc., 7650 W. Courtney Campbell Causeway, Tampa, Florida

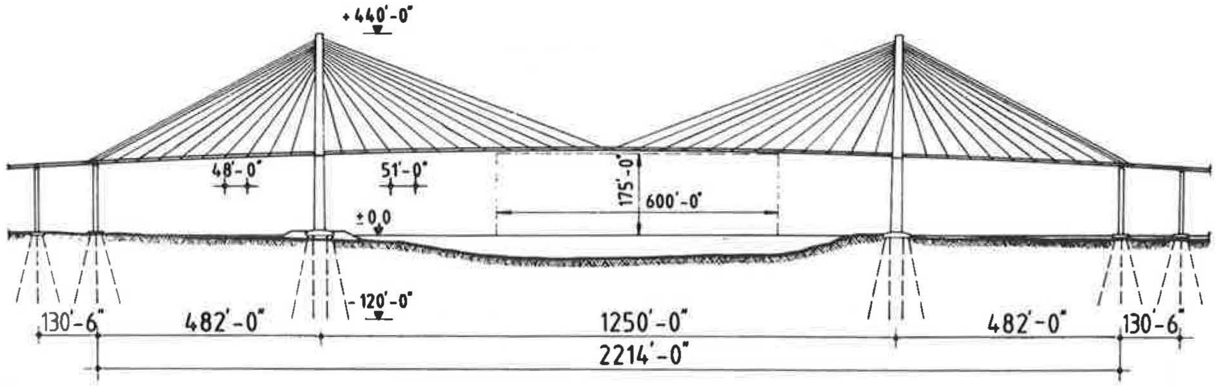

Fig. 1: General Layout

return period was determined as 110 mph at 30 ft elevation. By using an exponential function to reach the limiting wind speed of 200 mph at a 600 ft elevation, the basic design wind speed increases to 160 mph at deck level and 195 mph at the top of the towers. For the towers and piers an additional local gust factor of 7 % was applied.

The first 30 eigenfrequencies and corresponding mode shapes were calculated for a space frame. The important first natural frequencies in bending and torsion are f_B = 0.273 cps and f_T = 0.670 cps, resulting in a favorably high ratio of f_T/f_B = 2.45.

A high rotational superstructure girder stiffness for the torsionally weak open cross-section was achieved through the A-shaped upper tower legs. They form triangular space trusses with the towers as posts, the cables as tension diagonals and the girders as compression chords.Thus, providing much more rotational stiffness to the girder than H-shaped towerlegs would do.

The shape factors for the girder were determined in a wind tunnel from a section model with a scale of 1 : 96, Ref. (*1*).

In the final stage the traffic barrier and the safety fence create shape factors different from those during construction, see Fig. 2. Because of the interaction of the two girders different shape factors resulted also for the leeward and windward girder, see Table 1.

An analytical aerolastic analysis of the bridge in laminar and turbulent wind was then performed which rendered satisfactory results, Ref. (*2*).

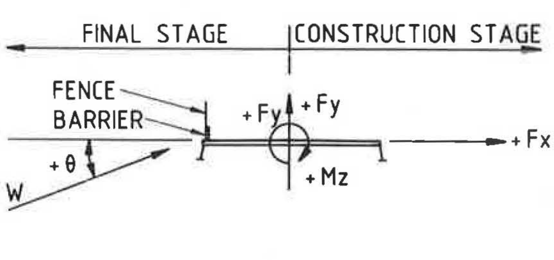

Fig. 2: Notation for Shape Factors

TABLE 1:
SHAPE FACTORS - MAIN SPAN BRIDGE DECK

Final Stage

Vertical Angle of Attack	Windward Bridge FX/Q	FY/Q	FZ/Q
- 2°	+ 14.537	- 32.062	39.392
0°	+ 14.075	- 21.724	- 46.300
2°	+ 14.028	- 9.044	- 113.503
Vertical Angle of Attack	**Leeward Bridge FX/Q**	**FY/Q**	**FZ/Q**
- 2°	+ 6.413	- 7.548	- 3.940
0°	+ 6.829	- 0.638	- 23.204
2°	+ 7.553	5.540	- 35.930

Windload = FT/Q x Q, with Q': Basic Static Wind Pressure

Construction Stage (W/O Fence and Barrier)

Vertical Angle of Attack	Windward Bridge FX/Q	FY/Q	FZ/Q
- 2°	+ 9.773	- 23.948	- 198.452
0°	+ 9.959	- 12.308	- 150.634
2°	+10.374	+ 2.357	- 118.323

Note:
Coefficients FX/Q and FY/Q are expressed in FT.
Coefficient MZ/Q is expressed in FT^2
For notation see Fig. 2

Because of the uncertainties involved with the interaction of the two girders, wind tunnel tests with a full bridge model were additionally executed, Ref. (*3*). The completed bridge was investigated as well as three construction stages, see Fig. 3. In both cases it was found that the aerodynamic interaction between the two beams by means of energy transfer to, and subsequent dissipation by, the leeward deck significantly enhanced the overall aerodynamic stability. The measured critical flutter wind speed was in excess of 150 mph for laminar flow. For an atmospheric turbulence of up to 12 % a peak to peak midspan amplitude of 5.42 ft is predicted for the completed stage, and of 8.22 ft for the critical construction stage just prior to reaching the anchor pier.

The computed results compared satisfactory with the wind tunnel measurements.

4 COMPOSITE GIRDER

4.1 Structural Details

The four lanes with full shoulders require a roadway width of 72 ft for each direction of travel. Two, three and four cable planes for supporting the roadway transversely were investigated. It was found that two independent girders - each supported by two outer cable planes, see Fig. 4, - are the most economical. Governing in this respect is the amount of steel required for the floor beams. It increases strongly if the transverse span length between cable planes is increased from 78 ft to 153 ft. The variation in the steel required for the main girders and the stay cables is comparatively minor.

The cross-section of an individual girder is shown in Fig. 5. It consists of a steel grid composite with a concrete roadway slab.

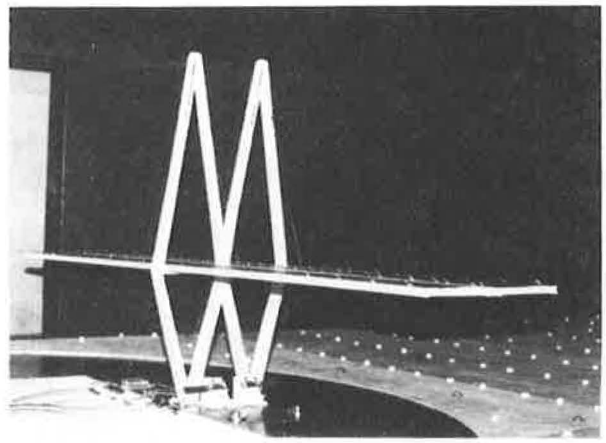

Fig. 3: Construction Stage Wind Tunnel Model prior to reaching the Anchor Pier, from Ref. (*3*)

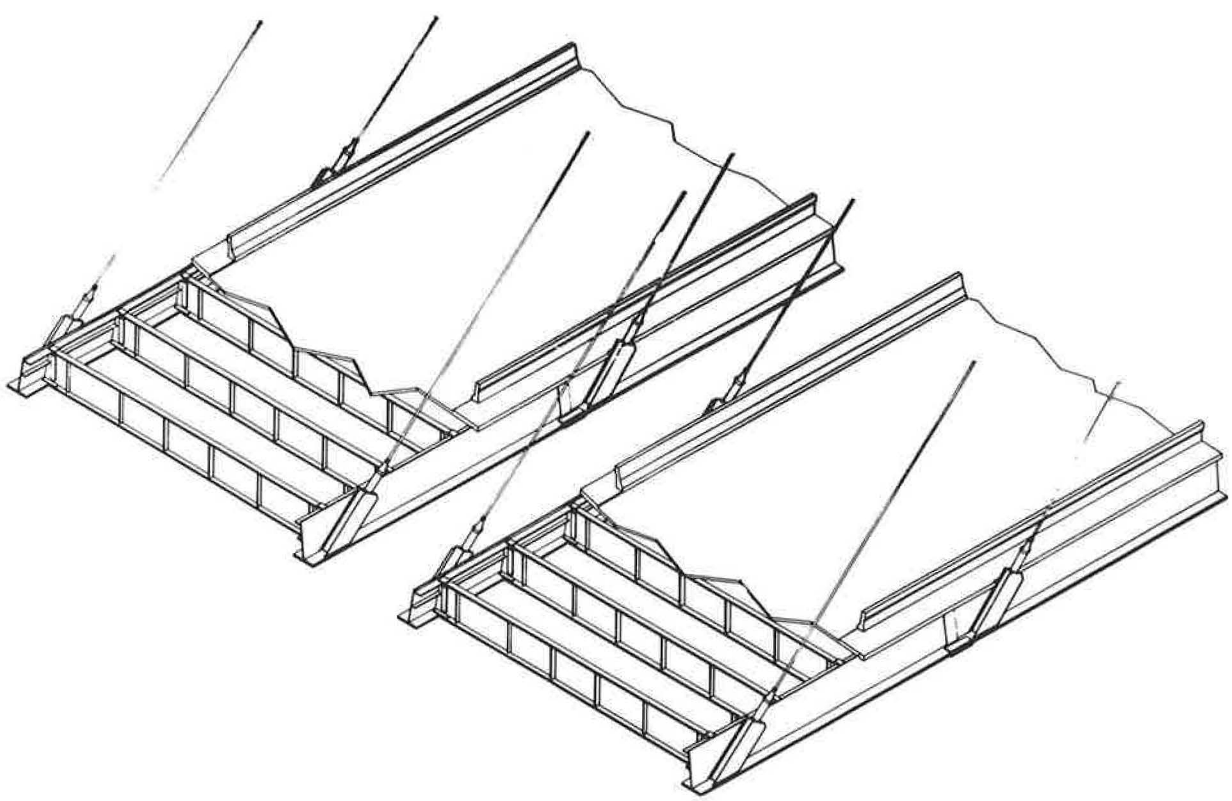

Fig. 4: Isometric View of Twin Girders

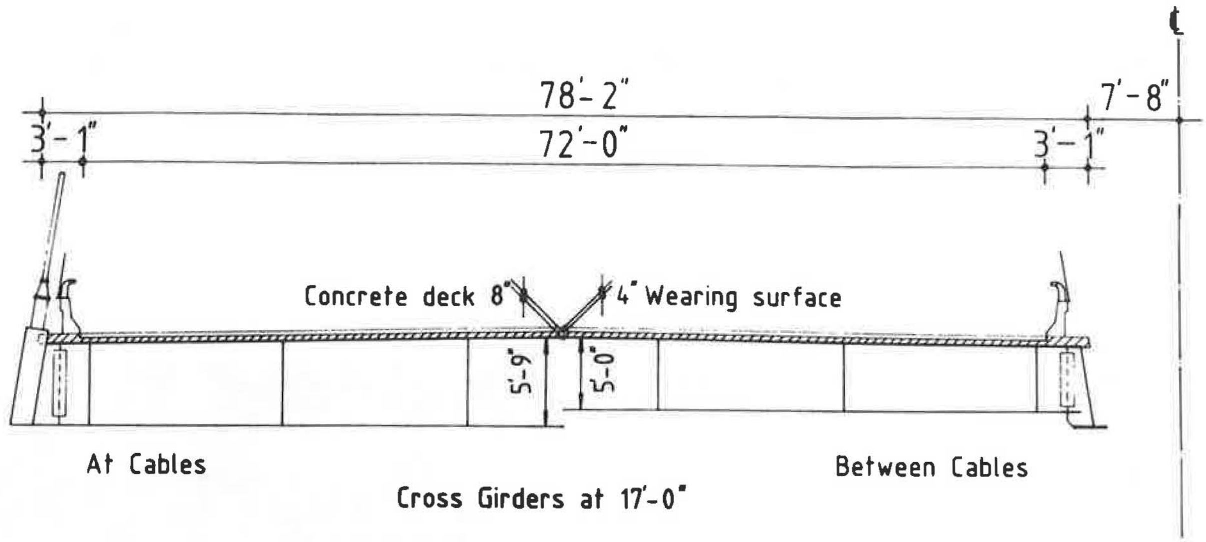

Fig. 5: Individual Girder Cross-Section

The outside main edge girders have one continuous longitudinal stiffener, see Fig. 6. The vertical stiffeners at 17 ft intervals are welded to the main girders, except for the regions of high moments near the center and the ends of the bridge where they are bolted to the bottom flanges due to fatigue. All floor beams are field bolted to the vertical stiffeners, see Figs. 5 and 6.

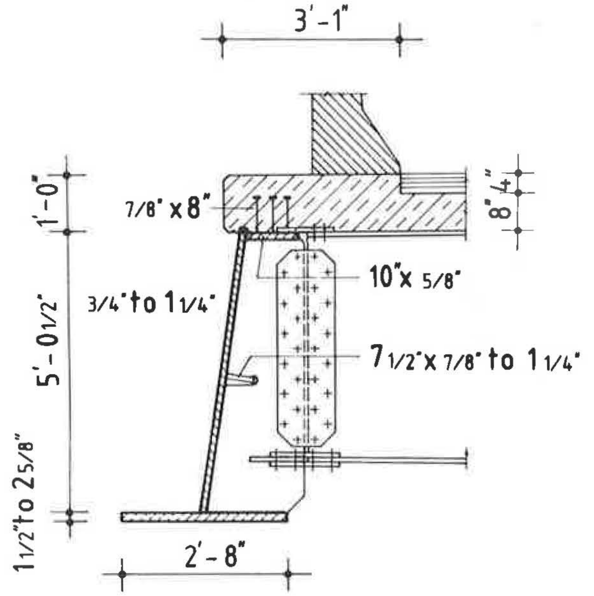

Fig. 6: Edge Girder Detail

The concrete deck is 8 in thick and has a 4 in reinforced concrete wearing surface. Such thick renewable wearing surface is used to safely protect the structural deck slab which would be more difficult to exchange. Longitudinal and transverse composite action is achieved by conventional shear studs. On the main girder top flange they are arranged in rows of three with a constant longitudinal spacing of 4 ½ in.

Crack control is achieved by a substantial amount of reinforcement with close spacing. At midspan where the compression is smallest 3/4 in diam. bars at 5 in spacing top and bottom (2.2%) are used longitudinally and transversely.

The cables are anchored in welded boxes bolted to the main girders, see Fig. 7. The eccentricity moment is carried by a force couple in compression to the roadway slab and in tension to the bottom flange of the full-depth floor beams.

4.2 Design

The overall girder forces under permanent loads were chosen similar to those for a beam rigidly supported at the cable anchorpoints, except for the midspan and end regions where a positive camber is introduced to provide additional compression in the roadway slab. Composite action for dead load was to be achieved by casting the roadway slab onto a continuously supported steel grid on ground . The deck was thus under compression in transverse direction also as top flange of a simply supported girder under dead load.

The shrinkage and creep values were calculated in accordance with the CEB-FIP Model Code (*4*), which resulted in approximately the following ratio of moduli of elasticity for permanent loads:

n_0 = 6.0 initially and for transient loads
n_1 = 12.5 at opening for traffic
n_∞ = 18.0 after creep has taken place
(W/C = 0.35, relat. humidity 75%, age of deck at installation 1 month).

The overall girder moments due to live load increased by up to 26% in the ultimate limit state due to non-linear effects of the rather slender deck with a main span to depth ratio of about 1 in 200, see Fig. 8 and (*5*). The edge girder shear studs are designed for the combined action of local

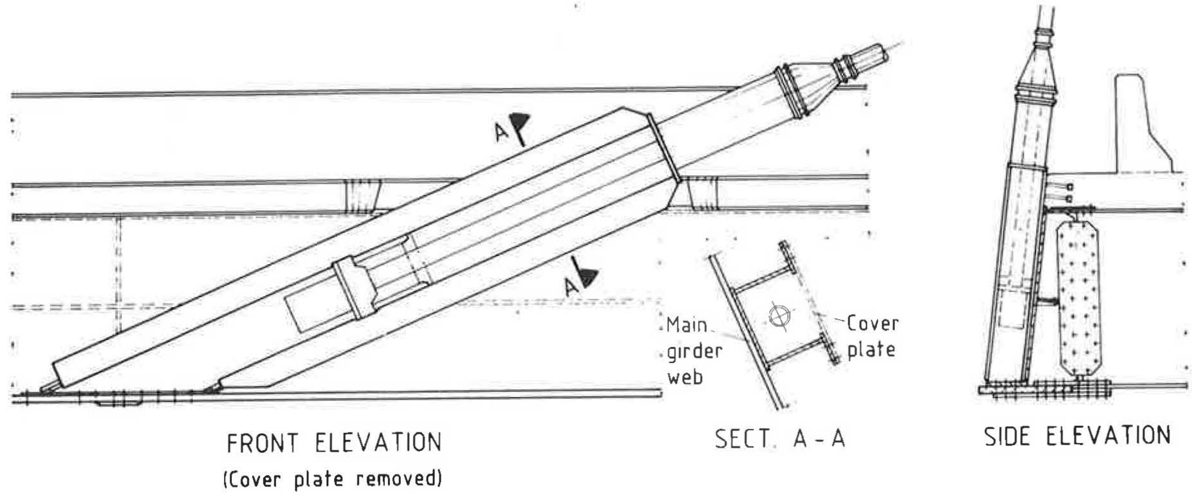

Fig. 7: Stay Cable Anchorage at Girder

and overall shear, and cable force introduction. An assumed limited amount of slip and plastic deformation of the studs in the ultimate limit state led to the uniform arrangement of shear studs over the length of the beam. The introduction of the shear force from the 12 in thick concrete edge beam into the regular 8 in thick slab (see Fig. 6) proved to be critical.

The sizing of the slab was governed by ultimate strength and crack control under service conditions. The plate girder stability was calculated in accordance with (*6*).

4.3 Contractor's Option

Instead of the proposed continuous roadway slab the contractor opted to use precast slabs connected by cast-in-place joints on top of the floor beams. Due to the resulting loss of composite action for dead load this required additional 1,412,000 lb of structural steel, or an increase of 17% from 24.8 lb/ft^2 to 28.9 lb/ft^2.

5 STAY CABLES

The stay cables were sized in accordance with the PTI-Recommendations (*7*), resulting in 19 to 61 7-wire 270 KSI strand with 0.6 in diam, see Fig. 9. They were specified as shop-fabricated parallel strand HiAm cables in PE-pipes with cement grout and a wrapping with a laminated Tedlar tape, see Ref. (*5*) for further details.

All stay cables are installed by jacking at the towerhead. They run through short steel pipes at the top and bottom anchorages, see Figs. 7 and 11. At the ends of these steel pipes annular neoprene washers are installed which act as dampers against cable vibrations.

The structure was designed to permit the exchange of any stay cable in conjunction with a reduction of live load to two lanes and reduced safety factors. Additionally, any stay cable may be accidentally severed under full live load without structural instability. Because of the close proximity of the backstay cables to one another, it was considered

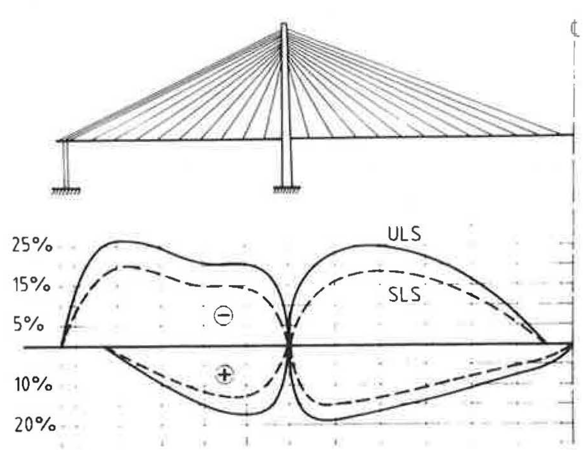

Fig. 8: Non-Linear increase of Live Load Movements

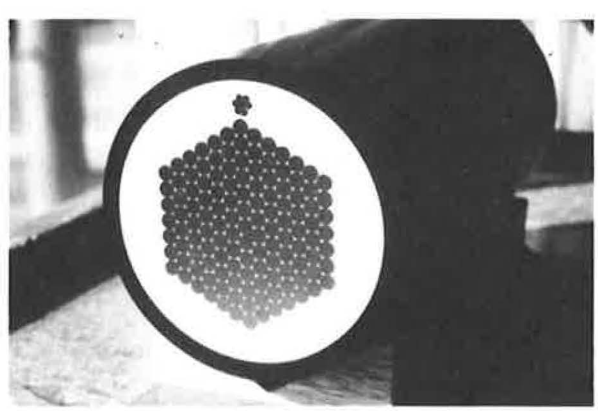

Fig. 9: Cross-Section of Stay Cable

prudent to protect them against simultaneous damage from a burning fuel truck. They are thus each surrounded by an additional larger PE-pipe, which reaches to 50 ft above the deck. The annular space of about 1 in between the inner and outer PE-pipes is filled with cement grout as an additional fire protection.

The contractor opted for site fabricated parallel strand cables with wedge anchorages in PE-pipes and cement grout.

6 TOWERS

The towers are shown in Fig. 10. Their legs and the tie beams underneath the decks have box sections with a minimum wall thickness of 12 in.

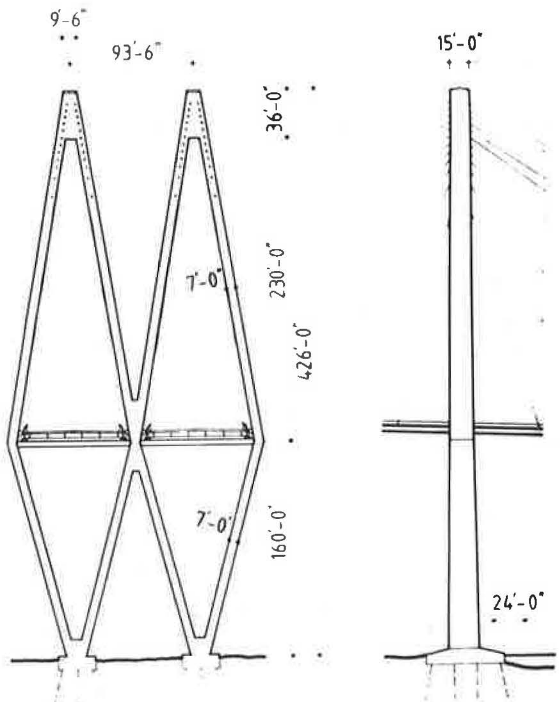

Fig. 10: Tower Layout

The double diamond shape is a natural progression from the twin decks. The A-frames on top of the decks reduce the rotations of the beam significantly by forming a triangular space frame, Ref. (*5*). By joining the two lower A-frames at deck level, a truss is created which carries the transverse wind loads in tension and compression to the two foundations. The transverse width of the towerlegs can thus be small. In longitudinal direction the tower legs act as cantilevers in bending, especially during construction. Those widths have thus to be significantly greater. The tie beams act as direct tension members and are fully post-tensioned against the outward thrust from the tower legs.

At the towerhead the stay cables pass through steel pipes embedded in the tower walls, see Fig. 11. They are individually anchored inside on steel bearing plates resting on concrete corbels. The horizontal cable components are tied back with alternating loop tendons so that each cable anchorage region is confined by the radial forces from the loops.

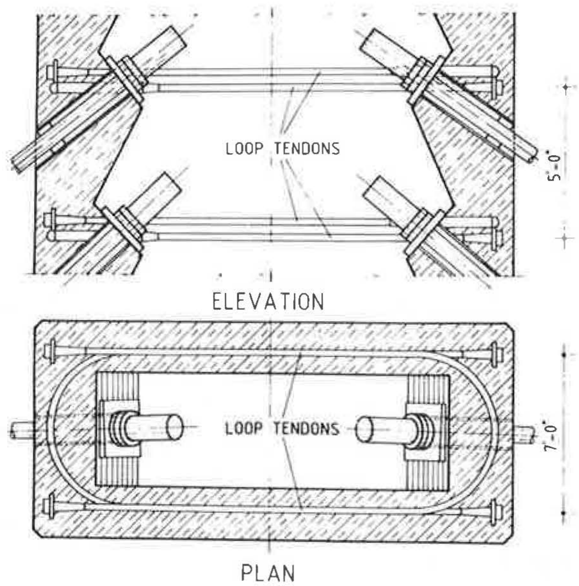

Fig. 11: Stay Cable Anchorage at Towerhead

7 CONSTRUCTION

From the four foundation alternates shown on the bid drawings the contractor opted to use 20 in. square precast prestressed concrete piles. 132 piles up to 136 ft long for each of the four tower foundations were driven from the artificial island on the LaPorte side and the existing levee on the Baytown side. 12 ft thick CIP pile caps form the basis for the towerlegs.

The towers are cast in 15 ft lifts with jumping forms, using trusses to support two cranes, see Fig. 12. All vertical reinforcement is spliced with mechanical couplers. They are squeezed hydraulically around the deformed bars.

Where the two inner legs meet, 220 rebars on each side cross one another. This cage was preassembled in a 45 ft high section, Fig. 13. Superplasticised concrete was vibrated into place without any significant honeycombing.

At about one half of the height above the deck temporary struts support the inclined legs against one another, Fig. 14.

The lower part of the 2nd tower is built with conventional jumping forms. First the inner legs are supported on trusses. Then the outer legs are tied back to the inner ones, Fig. 15.

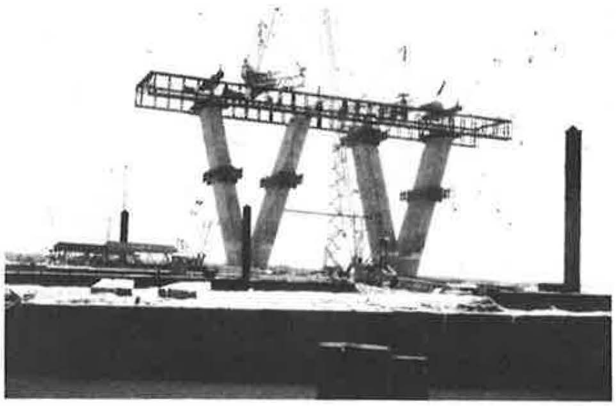

Fig. 12: First Tower Construction below Tie

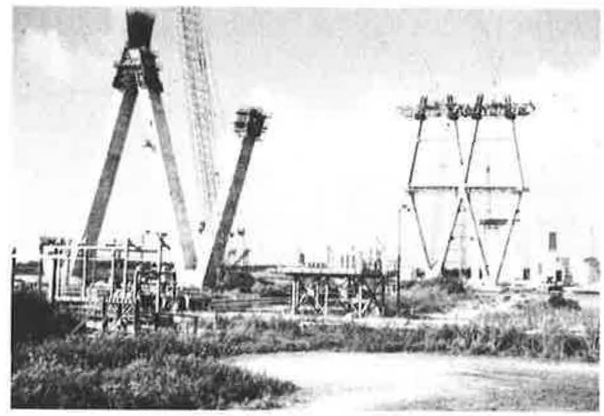

Fig. 15: 2nd Tower Construction below Tie

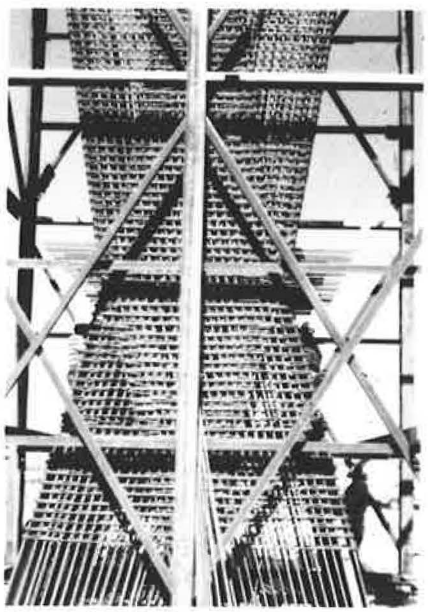

Fig. 13: Reinforcement Cage for Connection of inner Legs

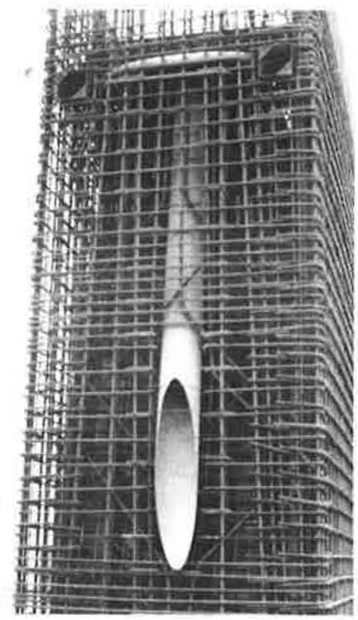

Fig. 16: Preassembled Section for Towerhead

Fig. 14: Construction of upper Tower Legs

Each lift in the anchorage zone is completely preassembled on ground, first the inner forms with the steel pipes, then the reinforcement is added, Fig. 16.

The about 5000 t of steel girders and their appurtenances are fabricated in South Africa. This is possible because the bridge is state-financed and the current Texas "Buy America" provisions were not yet law when the contract was signed in 1986, Ref. (*9*).

The stayed girders will be constructed by free cantilevering from the towers outwards. Construction started in 1987, and the bridge will be opened in late 1992.

The architectural model shown in Fig. 17 gives an impression how the completed bridge will look.

Fig. 17: Architectural Model

8 ACKNOWLEDGEMENT

Owner is the Texas State Department of Highways and Transportation. The cable-stayed main bridge was designed by Greiner, Inc., Tampa, Florida, in association with Leonhardt, Andrä and Partners GmbH, Stuttgart, Germany. Dr. Robert H. Scanlan served as aerodynamic consultant. Williams Brothers Construction Co., Inc., and Traylor Bros., Inc. (a joint venture), are the contractors. The stay cables are supplied by the VSL Corporation.

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