# Design and Construction of the Houston Ship Channel Bridge

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The Houston Ship Channel Bridge is a prime example of the "old" and the "new" in prestressed concrete bridge design. The approach spans use Texas DOT prestressed I-beams considered standard today. The main spans are prestressed segmental construction, new in the United States where all such bridges have been built within the last decade.

Further, in a variation from traditional bidding practice, "open specifications" were used to allow the low bidder to make modifications to the design compatible with his anticipated stressing techniques. The bidder was also permitted to modify the bridge cross section to suit his system of traveling forms.

This article will address the design, design modifications under the "open specs" system, and construction of the Houston Ship Channel Bridge including the 1500-ft (457 m) segmental concrete main structure which has the American record span of 750 ft (229 m).

This paper is adapted from one presented at the 1982 Segmental Bridge Conference (see footnote) in the session on construction which addressed construction problems and their field solutions.

**NOTE:** This paper is based on a presentation given at the Segmental Concrete Bridge Conference in Kansas City, Missouri, March 9-10, 1982. The Conference was sponsored by the Associated Reinforcing Bar Producers—CRSI, Federal Highway Administration, Portland Cement Association, Post-Tensioning Institute, and Prestressed Concrete Institute.

## LOCATION

Houston, Texas, is the third largest city in the United States, and one of the most unusual. The city had no natural access to the Gulf of Mexico until the

Describes the major design and construction highlights of the $60 million Houston Ship Channel Bridge, a prestressed concrete segmental structure having a record length 1500 ft (457 m) main span. The approach spans use Texas DOT standardized prestressed I-beams.

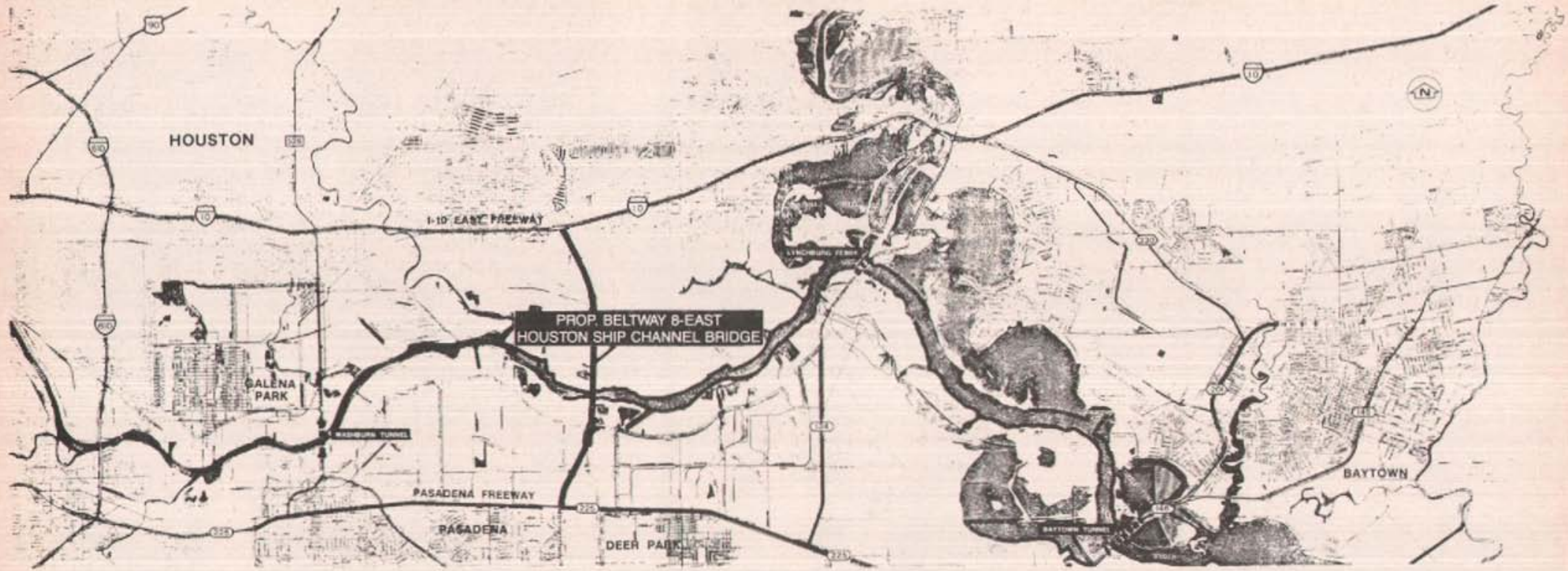



Fig. 1. Vicinity map showing Houston Ship Channel.

In February 1977, Howard Needles Tammen & Bergendoff (HNTB) presented a report to the Texas Turnpike Authority summarizing the results of a preliminary engineering review of a proposed crossing of the Houston Ship Channel. The proposed bridge would be located in eastern Harris County about 7 miles east of existing Interstate 610 on the Beltway 8 Alignment and would connect State Highway 225 with Interstate 10 (see map above).

Based on preliminary plans, reports, and other data prepared previously by the State Department of Highways and Public Transportation, the summary report included order-of-magnitude estimates of construction cost, and estimates of annual operational and maintenance expenses. Estimates of potential traffic and generated revenues were included and the feasibility of operating the project as a toll facility was reviewed.

These preliminary studies indicated an exceptionally high cross-channel travel demand. The anticipated toll revenues indicated probable financial feasibility when compared to estimated construction costs and operating expenses.

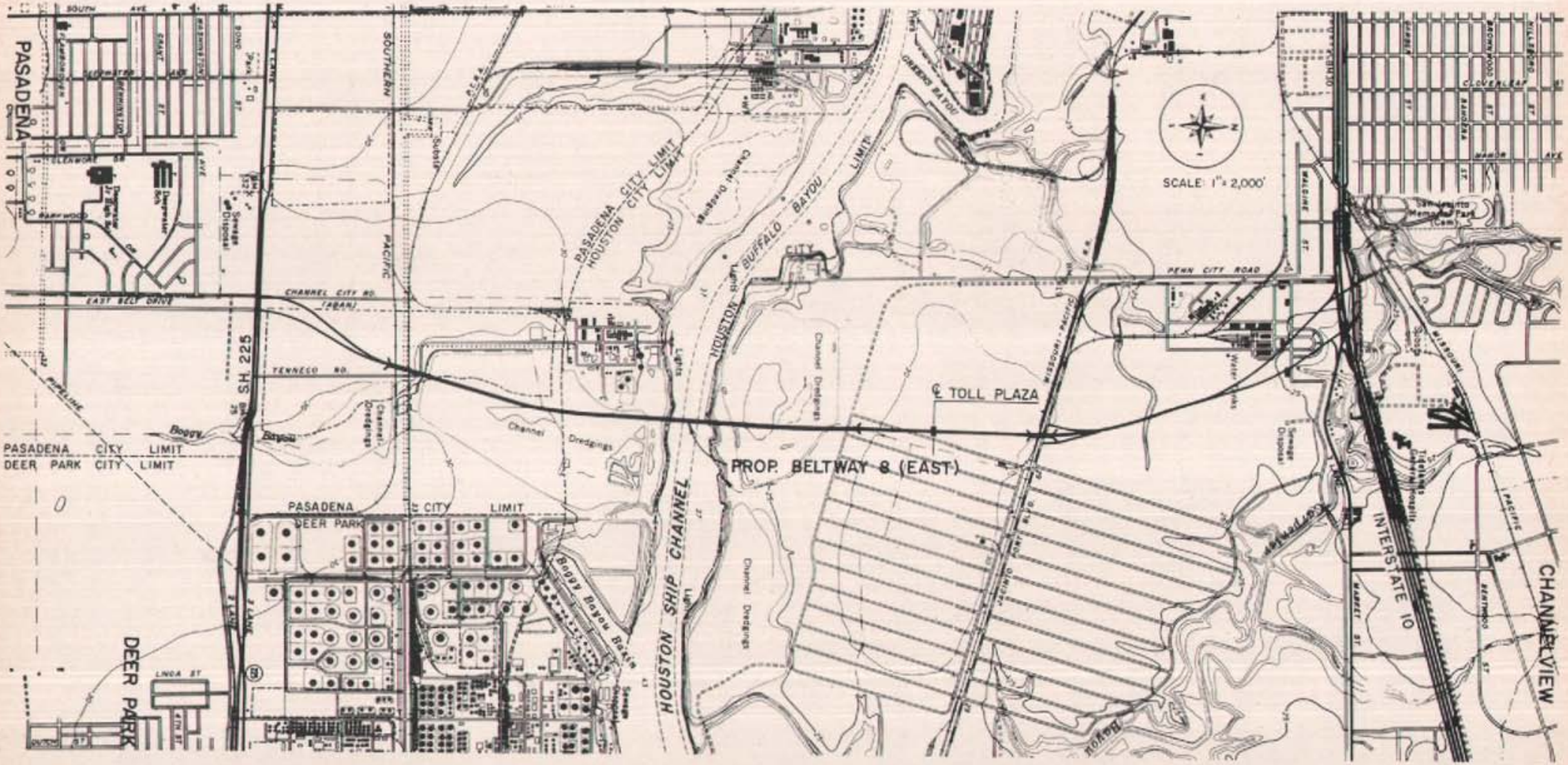



Fig. 2. Bridge location map.

Corps of Engineers widened and deepened Buffalo Bayou by dredging the 25 miles (40 km) to Galveston Bay. The job was completed in 1914 and Houston was officially established as an ocean-going port.

Today, the Houston Ship Channel runs 25 miles (40 km) along the bed of Buffalo Bayou and another 25 miles (40 km) across Galveston Bay to the Gulf of Mexico (see Fig. 1). The channel accommodates a wide variety of ocean-going ships and is the center of Houston's vast petrochemical industry. Before the new bridge was opened to traffic, severe traffic congestion existed in the area. An estimated increase of 250 vehicles a day poured onto Houston and Harris County streets and freeways. Much of the cargo destined for ships using the Houston Ship Channel were shipped by truck and heavy trucks comprised the bulk of the commercial traffic stream.

Prior to the opening of the Houston Ship Channel Bridge, there were four channel crossings — two tunnels, a ferry and the Interstate Route 610 Bridge. All crossings except the I-610 Bridge imposed severe restrictions on cargo and vehicle type permitted.

The Houston Ship Channel Bridge was envisioned as part of the outer circumferential route for the City of Houston (see Fig. 2). The 87.5-mile (141 km) roadway was planned to circle the city's central business district on a 12-mile (19 km) radius. By the end of the 1960's, about 36 miles (60 km) of the outer belt was either in place or under construction.

In 1972, the Texas Highway Department developed a preliminary design report for that portion of the beltway across the ship channel. In 1976, the Texas Turnpike Authority was authorized to investigate the possibility of constructing that 4.2-mile (6.8 km) segment as a toll facility.

In 1977 the Turnpike Authority commissioned HNTB, as its' consulting engineer, to do a preliminary feasibility study for the route as a toll road. The project was deemed to be feasible and the turnpike authority then requested HNTB to set up both a design and construction management team composed of local consultants and to set up formal design criteria to be utilized by the various consulting engineers in implementing the project's design concepts.

## PRELIMINARY DESIGN

In the preliminary design phase, eight structure types were studied for the main channel crossing. The purpose was to determine the best structure type or types to meet the conditions. A major consideration, of course, was cost, but constructability, interference with navigation, construction time, availability and price stability of materials and maintenance were also important considerations.

The various structure types (see Fig. 3) were:

1. Steel orthotropic deck box girder
2. Steel strutted box girder
3. Steel tied arch
4. Steel half-through arch
5. Steel cantilever through truss
6. Steel cable-stayed girder
7. Concrete cable-stayed girder
8. Segmental concrete prestressed girder

The concrete box girder, steel strutted girder and steel tied arch types were the three most competitive alternate designs. These three designs were compared in terms of interference with navigation during construction, long-term maintenance and construction scheduling.

Ultimately, the concrete box girder design was selected for the main spans. The construction required no restrictions on the channel, and could be completed in an estimated 4 months earlier. The structure will require a very low level of long-term maintenance. Also, when the design alterna-

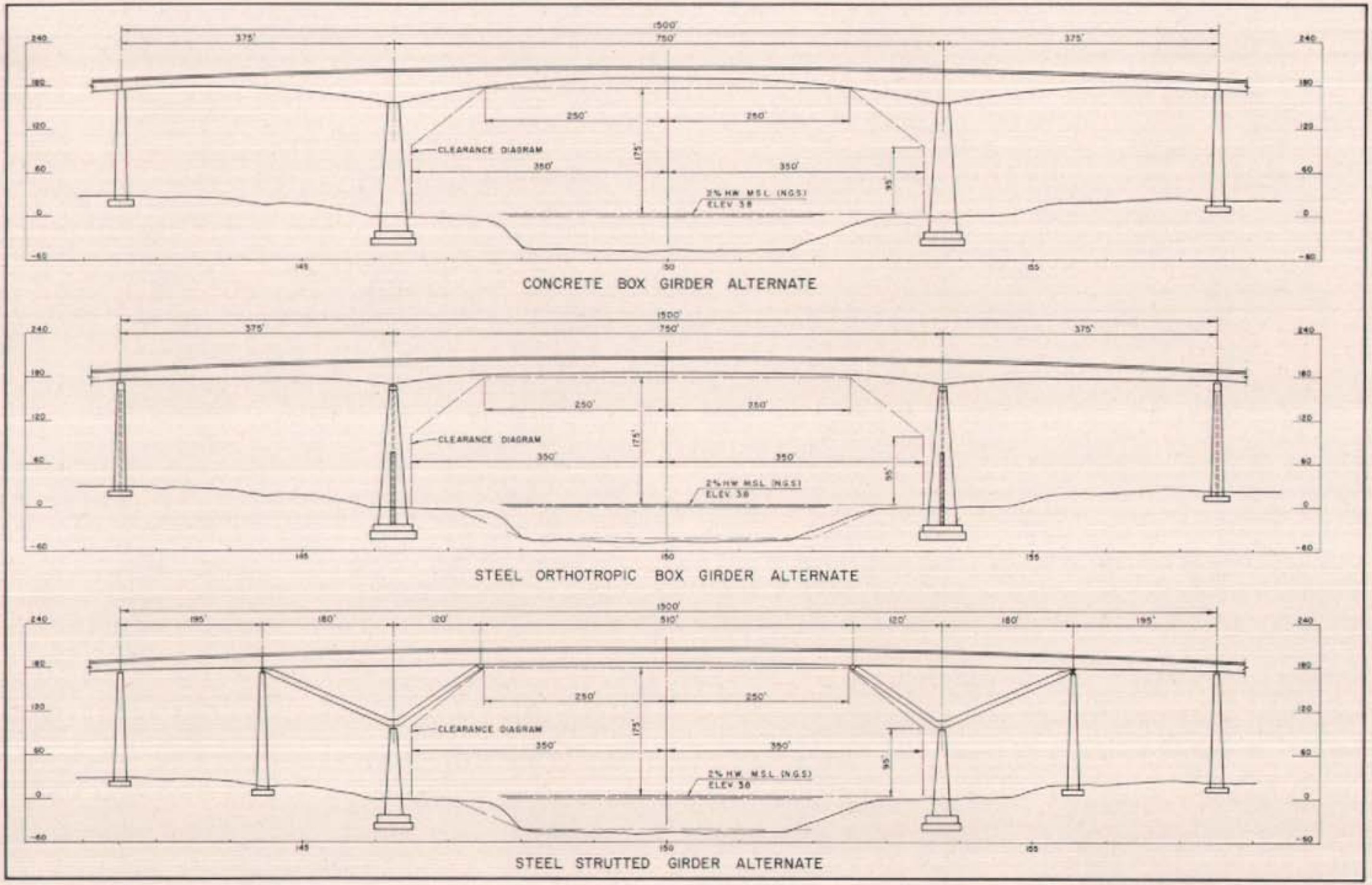
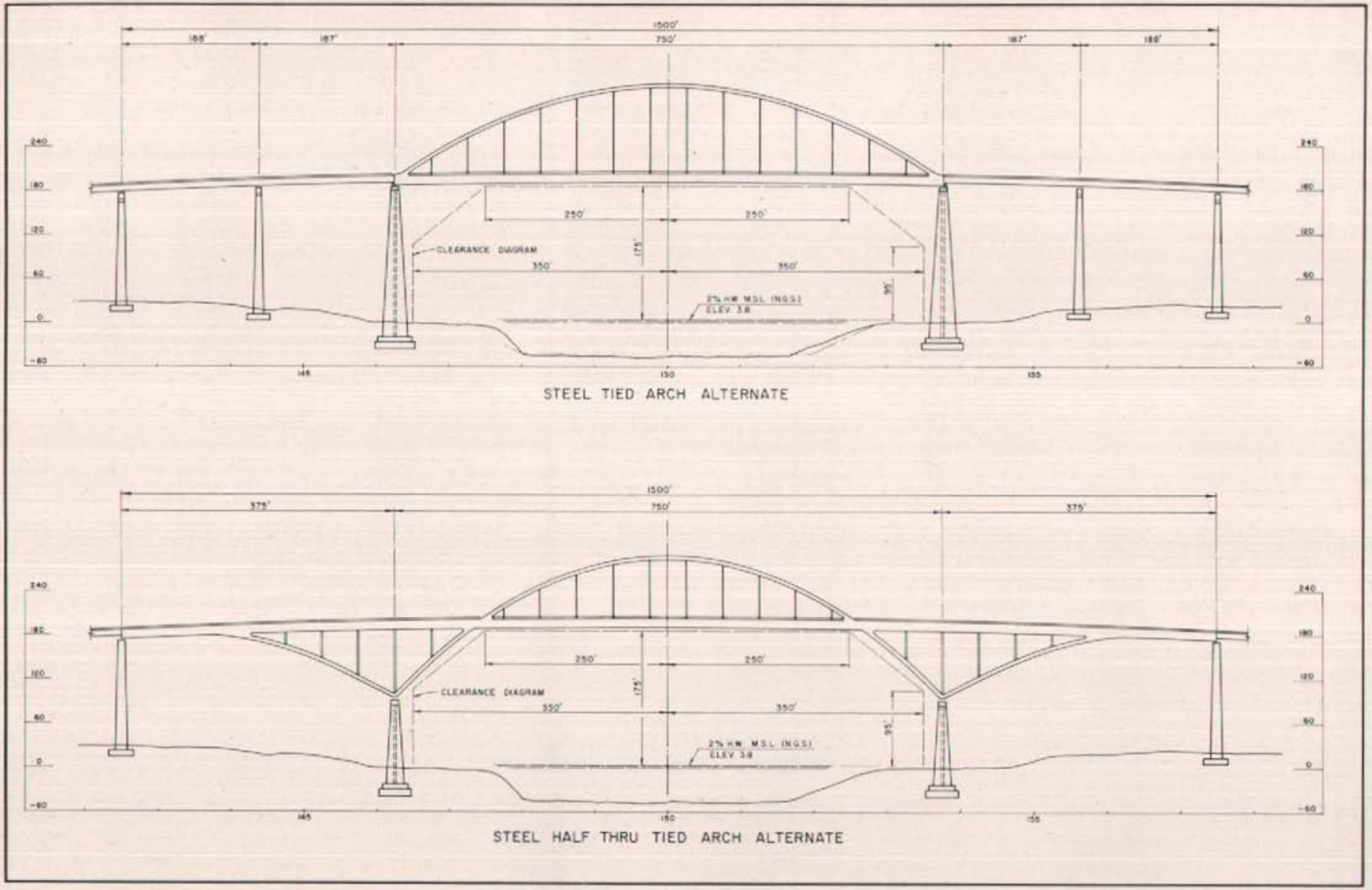



Fig. 3. Alternative structure types studied

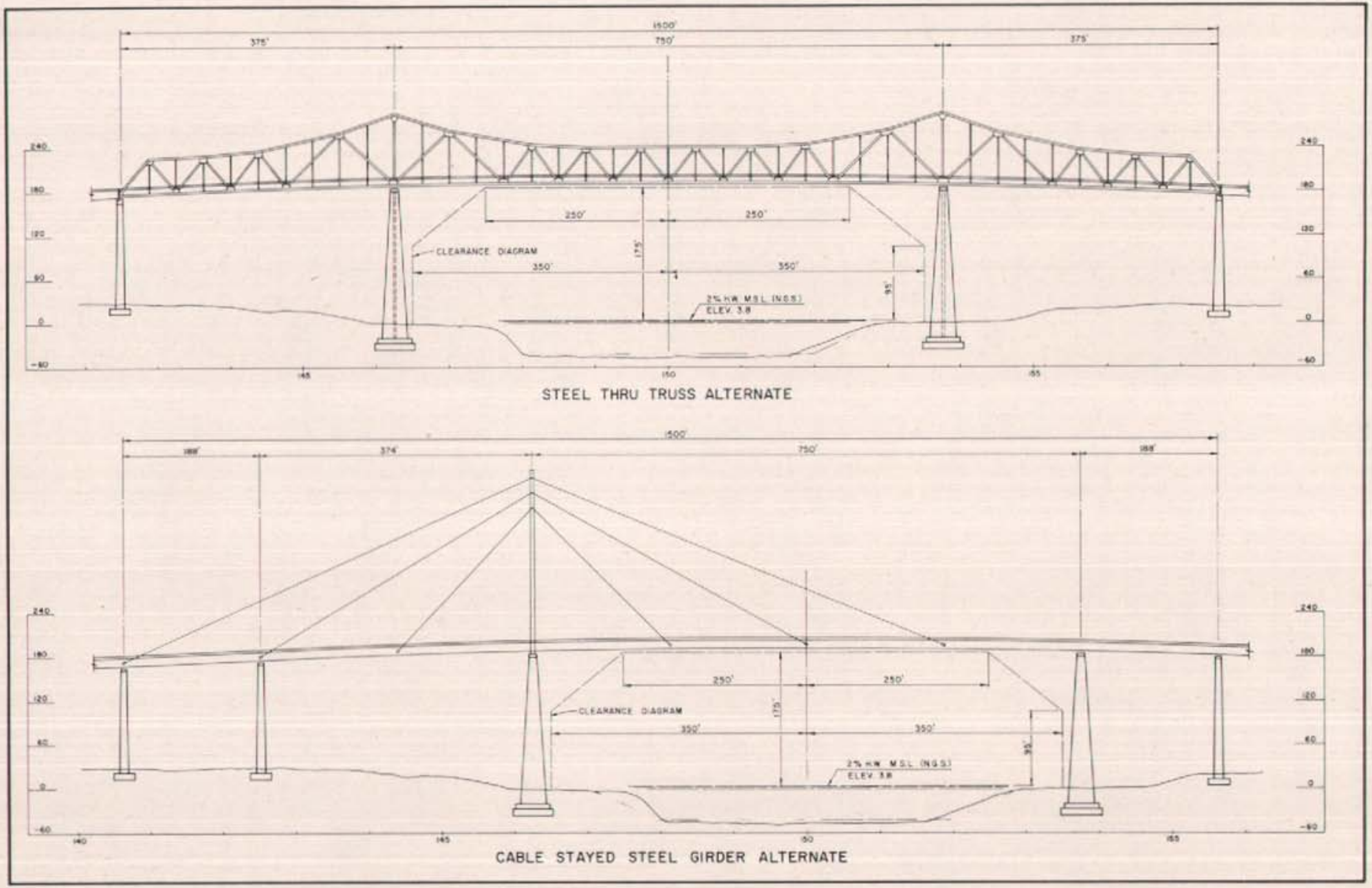
HOUSTON SHIP CHANNEL BRIDGE

(a) Segmental concrete prestressed box girder, steel orthotropic deck box girder and steel strutted box girder.




(b) Steel tied arch and steel half-through arch.

HOUSTON SHIP CHANNEL BRIDGE




(c) Steel cantilever through truss, concrete and steel cable-stayed girder.

HOUSTON SHIP CHANNEL BRIDGE

tives were considered, it was judged that the price of concrete was more stable than steel.

Steel delivery time was questionable with the volatile labor conditions at the time and the long-term maintenance costs were evaluated as being less with a concrete structure. Additional reasons involved the unskilled labor force in Houston and the fact that the balanced segmental construction technique permitted erection without adversely affecting traffic in the heavily traveled Houston Ship Channel.

## DESIGN

The design is predicated on an initial four-lane bridge with provisions for adding a parallel twin bridge in the future. At that time, the first four-lane, two-way bridge would be converted to three lanes one way with full shoulders.

Revenue and traffic projections indicate the initial bridge will reach capacity in about 8 years and be financially able to support construction of the parallel bridge. Right-of-way has been acquired in the first stage to accommodate the future bridge and roadway. The interchange connections, toll collection facilities and drainage have been designed to serve both stages of the project.

The lanes are split to accommodate construction of the future through lanes and connecting ramps with minimum interference to I-10 or the outer belt traffic. The parallel structure will be west of the initial bridge. The diamond interchange at I-10 will be a directional interchange when the project is expanded.

The approaches are 54 and 72 in. (1370 and 1830 mm) Texas DOT prestressed concrete I-beams of 94 to 120 ft (28.6 to 36.6 m) spans. The south approach is 6000 ft (1829 m) long, the north approach just under 3000 ft (914 m). The contractor was given the option to use precast prestressed deck forms which were designed to function compositely with a cast-in-place topping. This system was used on most of the approaches. The rest of the project comprises about 2 miles (3.2 km) of roadway and another 1300-ft (396 m) I-beam bridge. The total construction cost of the project is approximately $60,000,000. The main span bid was approximately $19,000,000.

A special study was made to ascertain the effect of higher fuel costs and fuel shortages on the revenue. With reasonable allowances for these items, the anticipated need is still sufficient to support the project.

The Houston Ship Channel is a busy waterway in this area. It serves for passage of all types of craft including ocean-going vessels. Because of this, the Coast Guard required a clear channel 700 ft (213.4 m) wide with vertical clearance over the central 500 ft (152.4 m) at 175 ft (53.3 m). This will provide two-way traffic as well as maneuvering room into the many dock areas. The main bridge is a three-span unit with the navigation span 750 ft (228.6 m) long.

It appears under such marine traffic conditions that it would be unfeasible and dangerous to obstruct the channel with falsework or erection bents. It was considered costly, risky and not too desirable to have erection barges in the navigation portion of the channel. This was a major consideration in the selection of the structure type.

Borings and geological studies indicated that the site was overlaid with a plastic, poorly bedded clay interbedded with lentils of sand. This led to a selection of piling foundations supported primarily by friction in cohesive type materials. The main span is supported on 24-in. (620 mm) diameter pipe piles, the approaches mostly on prestressed concrete piles.

For the past several decades, heavy pumping of the ground water in the Houston-Galveston region has resulted

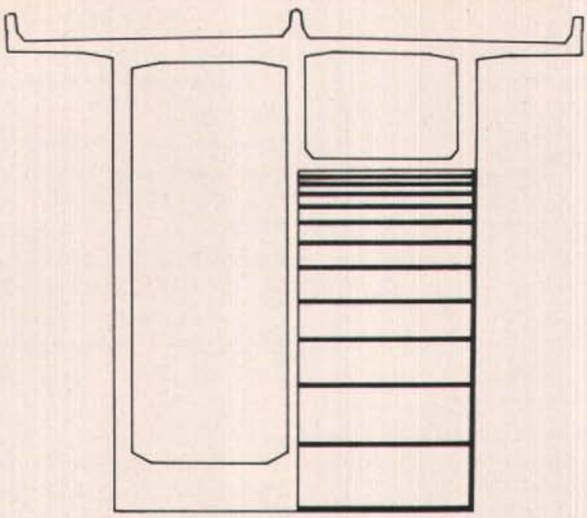



Fig. 4. Bridge cross section as designed.

in water level declines in wells of 200 to 375 ft (61 to 114 m). This has caused a general subsidence of the land over the region. The maximum subsidence has occurred at Pasadena, an incorporated area just southeast of the bridge. The subsidence contours for the maximum subsidence are almost centered on the bridge. A total subsidence of 7½ ft (2.29 m) was recorded between 1943 and 1973. From 1964 to 1973, the last 9 years of that period, subsidence was at a rate of 0.4 ft (0.12 m) per year.

Restrictions on pumping are now in effect and some attempt at return pumping into the wells has been made. The subsidence rate is expected to taper off, but will continue for some time. The design of the bridge and selection of structure types gave consideration to the anticipated differential subsidence over the life of the project. The design criteria provided for a differential movement of 9 in. (2290 mm) between the main and side piers. This covers the predicted differential subsidence plus foundation settlement, and will assure the safety of the bridge.

The plans prepared for bid provided a cross section with two 27-ft (8.23 m) roadways, a 2 ft 3 in. (0.68 m) New Jersey type median barrier and 1 ft 6 in. (0.46 m) wide parapets. The median barrier was designed to be removable in the future for conversion to one-way traffic. The main span is 750 ft (228.6 m), the side spans 375 ft (114.3 m). The bridge was designed for an HS-20 traffic loading and, although maximum operating speeds were projected to continue to be 55 miles per hr (88.5 km/hr) the design criterion was based on a higher design speed of 60 miles per hr (96.6 km/hr). The Houston design cross section consisted of a two-cell box with a slab overhang of 10 ft 7½ in. (3.24 m) and three 14-in. (0.36 m) wide webs spaced at 19 ft (5.79 m). A two-web alternate was studied but the balance of slab versus web tipped slightly toward three webs. The box was 12 ft (3.66 m) deep at midspan with a haunch at the piers 42 ft ⅞ in. (13.1 m) deep. The top slab varied from 8 to 18 in. (0.20 to 0.46 m); the bottom slab from 10 in. to 3 ft 9 in. (0.25 to 1.14 m) at the pier (Fig. 4).

The design provided for cantilever erection from both piers concurrently, with an imbalance of one segment. Because of the subsidence problem, a temporary hinge was provided in the side spans with a 95-ft (29 m) simple span from the hinge to the pier. The hinge would be "locked" during the cantilever erection with temporary prestressing tendons for the erection of those six segments. After closure with the landing at the pier, the prescribed positive moment prestressing would be placed and the proper reaction jacked in at the pier. The hinge would then be "unlocked" and the temporary negative moment tendons would be removed.

During design it was recognized that the hinges would be costly but they were considered less costly than designing a continuous structure for the settlement. Contractor costs in such a trade-off are difficult to forecast so the contractor was allowed the option under the redesign provisions to eliminate the hinge provided he designed for the full 9-in. (229 mm) differential settlement.

The outside parapets used a unique design. They were faced with the configuration of the New Jersey type barrier but were designed higher and stronger to contain an 80,000 lb (356 kN) truck traveling up to 50 miles per hr (80 km/hr.) The loading and design were established with the help of barrier performance studies made at the Texas Transportation Institute as well as data from the FHWA. The design adopted was based on the balanced cantilever method using prestressed segmented cast-in-place concrete construction.

The controlling condition for establishing the section dimensions was the cantilever moment. The superposition of other moments gave the final moment diagram to which a tendon arrangement was closely matched by anchoring some tendons at each segment. The stepped arrangement was achieved by providing empty ducts and laying tendons as needed.

Three sets of longitudinal tendons were used: cantilever tendons, span tendons and continuity tendons. The cantilever tendons carry the loads during construction. These stresses are "locked in" and remain except for redistribution due to creep and shrinkage.

The cantilever tendons were anchored at each segment in the top fillet of the webs and thus were completely encased when the next segment was placed.

The continuity and span tendons provide for positive moments that occur under live load and redistribution of moments. These were placed and stressed when the closure pours were made. At that time the structure became a three-span continuous frame.

Because the effect of long-term creep and shrinkage can only be estimated, it is current design practice on long-span bridges to make provision for "contingency" tendons. These were merely ducts through the diaphragms and anchor plates which will permit adding positive moment tendons in the future if desired. The concern is not for safety, but rather to correct excessive creep deflection, if desired.

As is customary, additional ducts were called for in the design. This provided for construction problems in case of a blocked duct or broken tendon.

The contractor provided for partial prestressing before the concrete had reached full design strength to allow him to advance his forms and speed up the cycle. This was done by the addition of prestressing bars. These bars became part of the design prestressing.

## CONTRACTOR'S DESIGN MODIFICATIONS

Because cost is the most critical item on a toll project, an attempt was made to give the contractor maximum flexibility in his bid. Many hours were

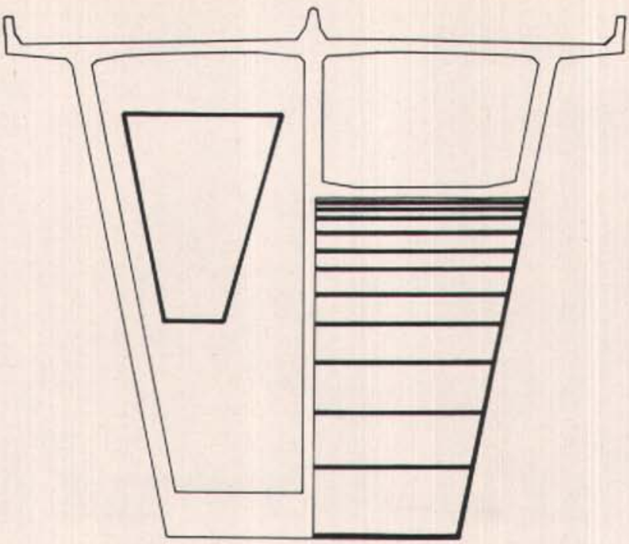



Fig. 5. Bridge cross section as modified by contractor.

spent to obtain the exact language in the contract documents to give this flexibility and still retain some control to exclude some possible unknown options not wanted. Value engineering, per se, was not compatible with the financial and bonding requirements but it was obtained, in effect, during the bidding stage.

In brief, the contractor was required to maintain the roadway cross section, the navigation clearances and the general type and shape of the bridge. He was allowed complete redesign as he so chose, subject to preliminary plans submitted with the bid and review of his final design.

Three bids were submitted. All three included designs which eliminated the hinge. The contractors felt the added material required to accommodate the settlement was not enough to offset the disruption and the continuity of the repetitive operation of casting segments.

The contractor changed the box shape in cross section to sloping webs. Some material savings resulted but the forming and steel fabrication and concrete placement became more complex. The variable depth required by the haunch made adjustment of the traveling forms more difficult and increased the cycle time somewhat (see Fig. 5).

The shape of the soffit haunches was changed from circular to parabolic with a deeper section at the piers but the general longitudinal shape differed too little to be visually apparent (see Fig. 6).

The most significant change was in the design strength of the concrete. The piers were designed for 5500 psi (37.9 MPa) and the superstructure for 6000 psi (41.4 MPa) versus 3600 and 5000 psi (24.8 and 34.5 MPa) in the original design.

The pier, a hollow box section, was redesigned, necessarily, to match the revised cross section. Because of the increased concrete strength the contractor decreased the wall thickness. This again provided substantial material savings but made placement of reinforcing steel and concrete difficult

Fig. 6. Artist's rendition of Houston Ship Channel Bridge.

and consolidation virtually impossible. Changes in the concrete mix and the use of supplementary external vibration were necessary.

The contractor elected to use partial longitudinal prestress of the segment to permit advancing the form travelers earlier. The partial prestress was provided by bars, the full prestress by tendons added later. To reduce the cycle time the partial prestress required 4000 psi (27.6 MPa) concrete in 40 hours. This in turn required a rich mix with Type III cement (high early strength). This combination of amount and type of cement, low water-cement ratio and ambient temperature made pumping impossible and placing by any means very difficult.

Other changes in redesign were minor and incidental to those described. The segment lengths, for example, were adjusted to fit the capacity of the contractor-designed travelers.

Bids were taken for the main span in March, 1979. Low bidder for this contract at $19.6 million was Williams Brothers Construction Company, Houston, Texas with Prescon Corporation, San Antonio, Texas, as the superstructure subcontractor. Prescon Corporation's engineer for the proposed design modifications was Figg & Muller Engineering, Inc., Tallahassee, Florida. Bids were taken for all the approaches and remaining portions of the project within a few months.

## CONSTRUCTION OF BRIDGE APPROACH SPANS

Alternate modes of construction were permitted on the foundations, either steel piles, concrete piles or drilled shafts. With the exception of the low spans on the south approach, where the drilled shaft alternate was part of the low bid, the prestressed concrete pile alternate was chosen by all the contractors. National Soils Service, Houston, Texas, performed the soils analysis. Their recommendation as to embedment length was shown on the plans.

In addition, each contract included provisions for test piles. The use of test piles to determine the actual bearing capacity of the piles resulted in the reduction of pile length of at least 25 percent. Breakage of piles was under 2

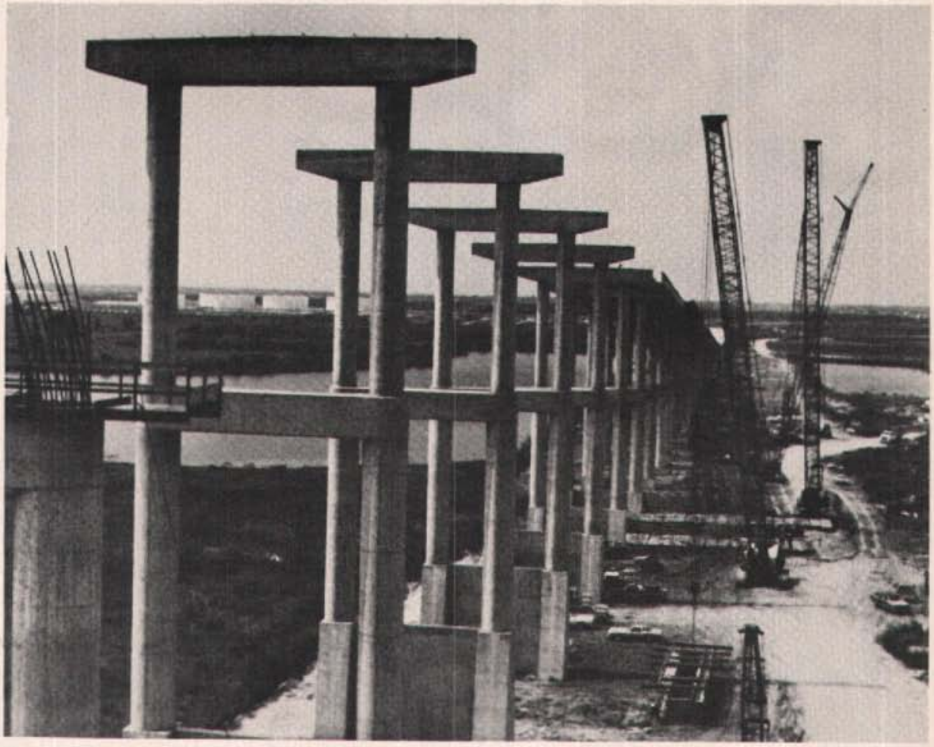


Fig. 7. Piers on land approaches.

percent for the overall project. The drilled shaft construction employed 8-ft (2.44 m) diameter drilled shafts. Approximately 20 percent had to be tremied, with the remainder concreted in the dry.

The piers were built in up to three tiers, as shown in Fig. 7. For purposes of economy these piers were made the same from the top down. The upper tier consisted of the pier cap and approximately 55 ft (16.8 m) of column. The middle tier employed a diaphragm and another 55 ft (16.8 m) of column. The lower tier employed trapezoidal columns connected by a web wall. Thus, a 40-ft (12.2 m) high pier would have a pier cap and straight columns. An 80-ft (24.4 m) pier would have the entire upper tier, the diaphragm and the remaining columns.

Once the pier reached a height in excess of 60 ft (18.3 m), the entire upper portion remained the same. Similarly, when the height exceeded 120 ft (36.6 m) the entire upper portion remained the same. This allowed the contractor's maximum economy in reusing the forms which was reflected in their bids.

Superstructure construction started with beam erection. On this project 54-in. (1.37 m) AASHTO type beams for the 120-ft (36.6 m) spans were set on elastomeric bearing pads. Each contractor had his own method of setting beams. Most contractors utilized two cranes in setting beams, as shown in Fig. 8. One contractor developed a delta frame consisting of tubular steel for the delta and flat plates for stiffeners. This approximately 25-ft (7.6 m) lifting frame permitted him to lift most of the beams with a single crane. Steel diaphragms were used to provide stiffness and lateral support during construction.

Fig. 8. Placing AASHTO type beams on land approaches.

Similarly, some contractors elected to use precast prestressed concrete deck panels in lieu of conventionally placing the deck (see Fig. 9). These 4-in. (102 mm) panels were prefabricated on site and put into place with a 3½-in. (89 mm) cast-in-place deck placed on top of them. This composite deck panel eliminated the wood formwork. Finally, after the decks were placed, the median and parapets were placed. The parapet was a conventional "Jersey Barrier" type. It was, however, 1 ft (0.3 m) higher than the conventional barrier and heavily reinforced. On some contracts this barrier was cast in place in conventional forms, on others it was slipformed. The median was either precast, poured in place in forms or slipformed, depending on the contractor.

## MAIN BRIDGE CONSTRUCTION

### Foundation

While foundation design modifications were not permitted in the specifications, this bridge, like most bridges built over a waterway, had some difficulty during foundation construction. The main channel piers are supported by 85 x 79 x 15 ft (25.9 x 24.1 x 4.6 m) thick footings which are founded on 255 — 24-ft (6.1 m) diameter ½-in. (12.7 mm) thick wall, open ended steel pipe piles.

The specifications anticipated problems with pile heave caused by driving through and into various clay layers. Redriving of all heaved piles was a specification requirement. The contractor's cofferdam consisted of PZ 36 sheet piles driven to a tip elevation of −50 ft (−15.2 m). Pile tip elevation was −110 ft (−33.5 m) with the bottom of footing elevation being −25 ft (−7.6 m). Due to the stability of the various clay layers encountered, no tremie seal was required. This permitted pile driving operations to take place after dewatering and excavation.

The anticipated heaving did, in fact, occur. The piles heaved as much as 18 in. (0.46 m) and caused as much as 9 in. (0.23 m) of differential deflection in the wales of the cofferdam. The eccentric-

Fig. 9. Precast prestressed concrete deck panels used on land approaches.

ity of the loading caused stresses in excess of 30 ksi (206.7 MPa) on the wales. In order to assure a safe working environment, an additional wale was added to the cofferdam.

Redriving of heaved piles could have been a never-ending operation. Because the area influenced by each pile was so large and because the contractor's superstructure modifications lightened the loads on the piles by about 10 percent, the contractor was advised that he could test load the piles in order to determine their bearing capacity in the heaved condition. The contractor elected to follow this option. The test load revealed a carrying capacity of the piles to be 350 tons (3100 kN), as opposed to the nominal 140 tons (1245 kN) design load. As the factor of safety was greater than two, no redriving was necessary.

### Pier Shaft

The contractor's modifications to the superstructure involved sloping webs rather than a rectangular box. This change required that the pier shaft also be revised to take the new box girder shape. These modifications were submitted with the bid package. The specified strength of the concrete in the original design was 3600 psi (24.8 MPa) while the modified design required 5500-psi (37.9 MPa) concrete. This increased strength, while more costly and difficult to attain, enabled the contractor to reduce the wall thickness of the land piers from 24 to 16 in. (610 to 406 mm) and reduce the transverse wall of the channel pier from 33 to 24 in. (838 to 610 mm). The reduction in the wall thickness required the contractor to redetail the reinforcing steel for the pier shaft.

This decision, while providing a savings in material cost, greatly increased the difficulty of placing the concrete. The contractor elected to place the wall lifts for the pier shafts in 27-ft (8.2 m) increments. The relatively thin walls on the land piers caused major concrete placing problems. The amount of room remaining after placing the reinforcing steel cages only permitted a 6-in. (152 mm) chute to be utilized for concrete placement. There was insufficient space for a man to fit inside the walls to vibrate the concrete.

This constraint resulted in all the concrete placed being vibrated from a height of approximately 30 ft (9.2 m). In order to make the system work the contractor then utilized a combination of external form vibration and high cycle internal vibrators. This involved strengthening up the forms to take the additional loads caused by the external form vibrators. This combination aided in solving the placing problem.

The corner reinforcement was difficult to place. Steel from the walls intersected each other and with the addition of corner reinforcing, caused a good deal of congestion. It became necessary to shift the splice bars in order to allow concrete to bond around the steel. The specified corner bars, a box bar with tails extending through each wall, were changed to double J bars for placing purposes. Upon completion of all of these modifications to the details, the concrete could be placed in the walls in a proper manner in order to produce well consolidated concrete.

## Pier Table

As noted, the contractor's design modifications changed the rectangular webs to sloping webs for the box girder construction. In addition, to take advantage of the design modification provisions of the specifications, he specified 6000-psi (41.3 MPa) concrete in lieu of the 5000-psi (34.5 MPa) concrete required. To eliminate partial post-tensioning a high early strength of 4000 psi (27.6 MPa) in 40 hours was also specified. All of these decisions had major ramifications concerning the construction and, in fact, the decision concerning the high early strength of the concrete resulting in significant placing difficulties for the concrete.

To achieve the high early strength, the contractor specified a Type III cement mix that proved to be highly reactive. The first placements made with this mix involved the 180 cu yds (137.7 $m^3$) bottom slab of each of the two pier tables. The slabs were pumped with difficulty but were successfully completed. The next placement involved the first wall lift of the 48 ft (14.6 m) pier table. In attempting to place concrete in the 180 lineal ft (54.9 m) of walls that rest on the bottom slab, two pumps malfunctioned and the placement had to be aborted. The work was halted for a period of approximately 2 months while various experiments were made in order to modify the concrete mix design so that a workable concrete could be found that would produce an acceptable product.

The basic problem experienced was the rapid slump loss versus time which permitted only 45 minutes from the time the concrete was batched to when it was placed. With an off-site batch plant 20 to 30 minutes away from the site, this left insufficient time to place the concrete. Various trial batches were made with different proportions for the ingredients. The contractor, the testing laboratory (Southwestern Laboratories, Houston, Texas) and HNTB independently investigated the problem. The ultimate mix design was a joint effort of these three parties.

This solution consisted of a superplasticizer as well as a more powerful retarder in order to better control the mix design. The intent initially was to leave the water-cement ratio as designed at 0.39 and increase the slump of the concrete utilizing the superplasticizer to give additional time to handle the concrete. This worked extremely well and 9 to 9½ in. (229 to 241 mm) slump concrete was utilized at a slight increase in strength to place the remainder of the walls.

A fourth and final placement in the pier table was the top slab. Ninety-two 4-in. (102 mm) longitudinal ducts went through the top slabs as well as the 14

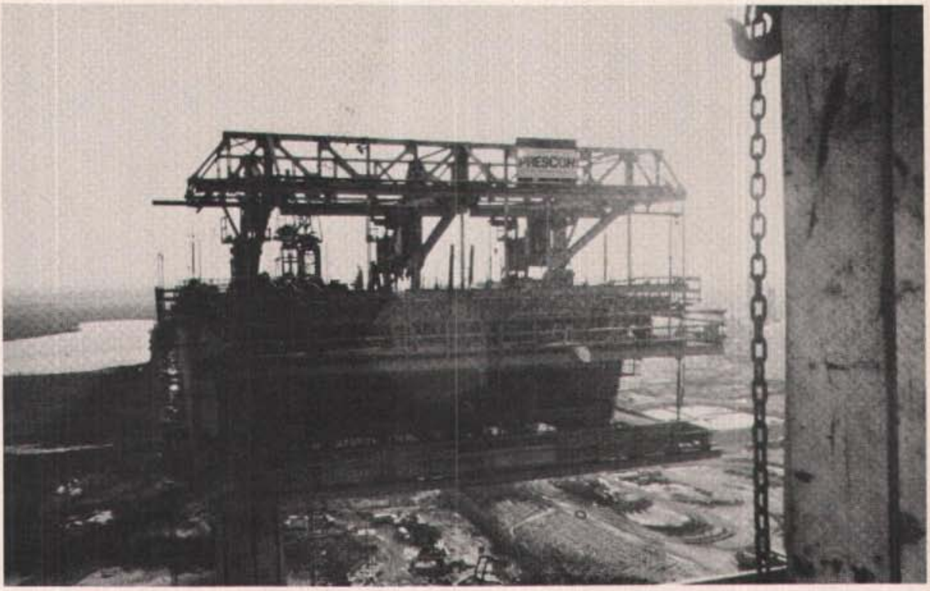


Fig. 10. Traveler set ready for concrete placement.

transverse ducts. These ducts were utilized for the 270-ksi (1860 MPa) 0.6-in. (15.24 mm) 7-wire, low-relaxation strands. In addition, vertical post-tensioning in the web walls had to be placed along with blockouts to make room for the jacks over the center. The contractor reduced the top slab from 18 to 10 in. (152 to 254 mm). The contractor's redesign failed to take into account the actual diameter of the elements being used and when the materials were placed, an 11⅛-in. (280 mm) deck would be required.

The solution to this problem was to slightly reduce the clearance requirements and use a 10½-in. (267 mm) slab. It is imperative that design engineers take into account the actual dimension of material utilized in this type of construction. Just as important is to leave sufficient space for fabrication and placing tolerances.

## Traveler Erection

The contractor designed the travelers for the project. The original concept indicated a 240,000-lb (1070 kN) traveler capable of supporting a 400,000-lb (1780 kN) concrete load. Problems developed in deflections and the alignment beams. After the travelers were modified their weight was increased to approximately 315,000 lbs (1400 kN). Due to the designed configuration of the traveler, it was approximately 6 in. (152 mm) too long and therefore both travelers could not be placed on the pier table simultaneously.

This problem was solved by setting the first traveler in its final pour position. After the concrete was placed and the segment stressed, this traveler was moved. At that point, the second traveler, which had been erected beyond its final position was pulled back into its pour position thus allowing the work to continue. Once the contractor was able to establish a working cycle he was able to turn over each set of forms in as little as 5 days. This rapid rate of progress allowed a very efficient working system to be developed. The travelers, as field modified, served the project very well (see Fig. 10).

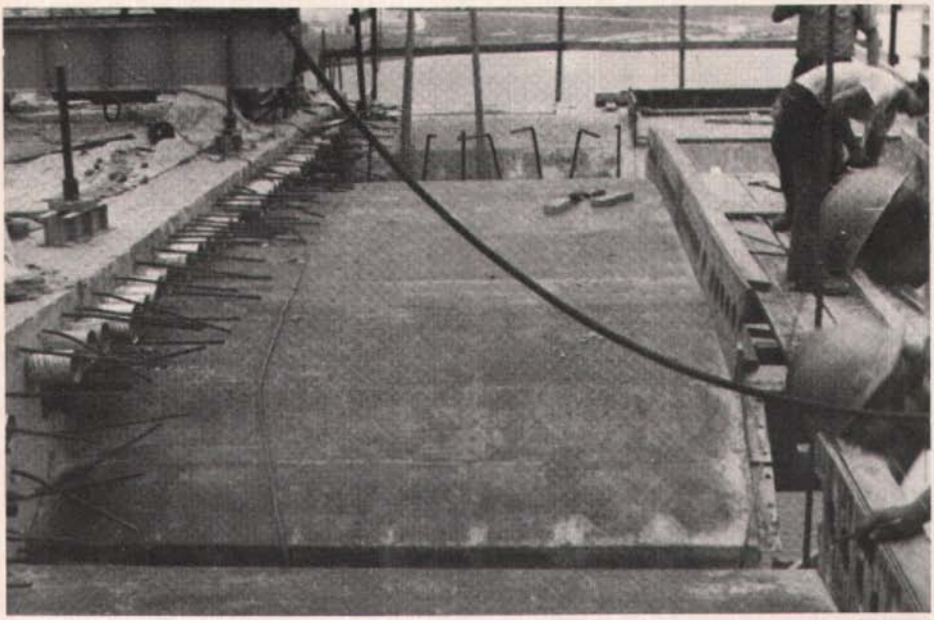


Fig. 11. Top slab formwork.

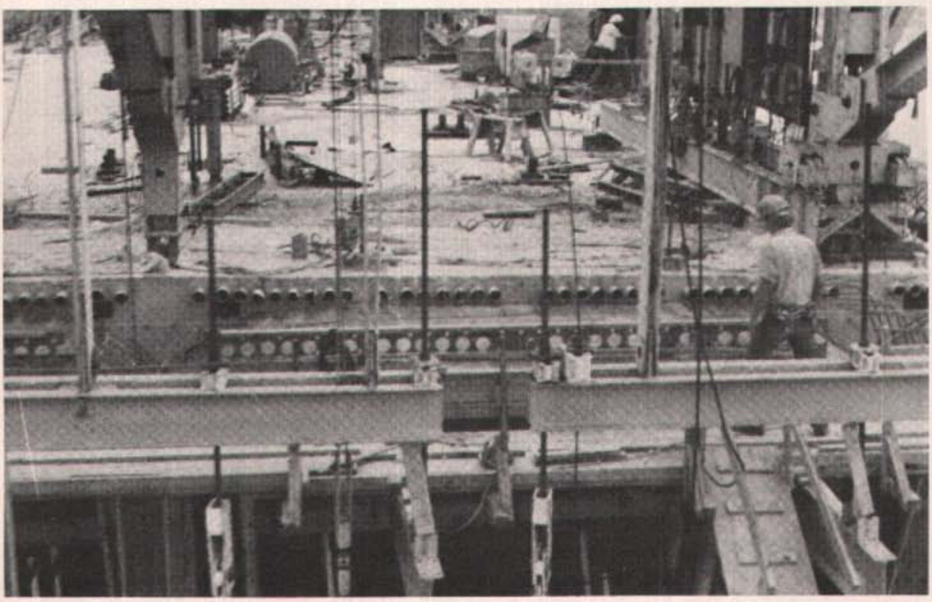


Fig. 12. Bulkhead for extension of top slab.

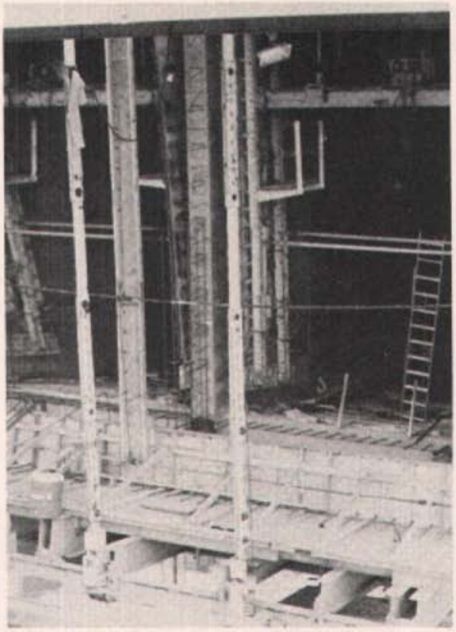


Fig. 13. Webwall forms.

## Segmental Superstructure

Segmental construction for the project involved very exacting quality control by both the contractor and the inspection team. This part of the project can be divided into a number of distinct parts.

**1. Form Alignment and Construction.** The contractor's decision to change the rectangular webs for the outside of the box to sloping webs provided a savings in material at a cost of additional labor (see Figs. 11, 12 and 13). Because the soffit of the structure is a third-order curve, the walls are cut at different angles; this makes the forming of the interior walls more difficult.

In addition, the thickness of the exterior walls was given as horizontal and not normal to the slope. This simple fact caused problems during erection of the forms because the natural tendency of the workmen is to measure normal to the forms. Support and alignment of sloping webs is a complex item of construction. The changing width of the bottom slab required changes in the transverse dimension of the soffit form every time the traveler was moved (see Fig. 14).

**2. Construction of Embedded Items.** The reinforcing steel and the ducts that are placed for future insertion of the strands must be located exactly as detailed on the plans for structural purposes (see Fig. 15). Each segment placed is distinctly different from the previously placed segment. Exacting quality control is necessary. Problems that arose included interference and conflicting call-outs on the plans. A decision had to be made on which embedded items had priority. Close liaison with HNTB and the contractor's engineer was established and the final decision on the priorities was set by the designers.

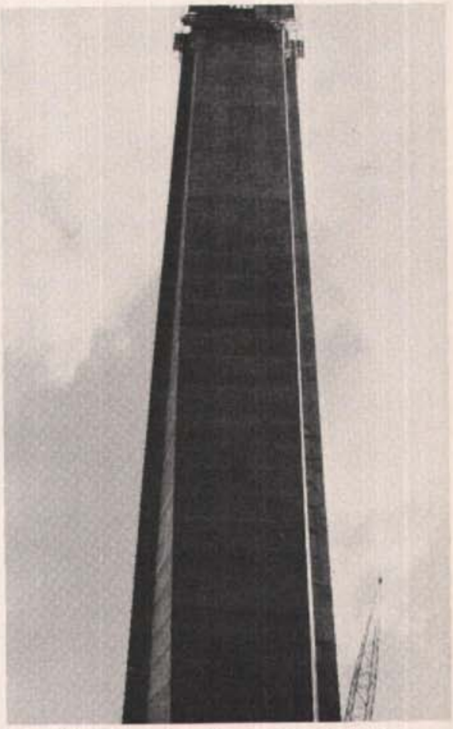


Fig. 14. Changing dimension of width of bottom slab (looking up).

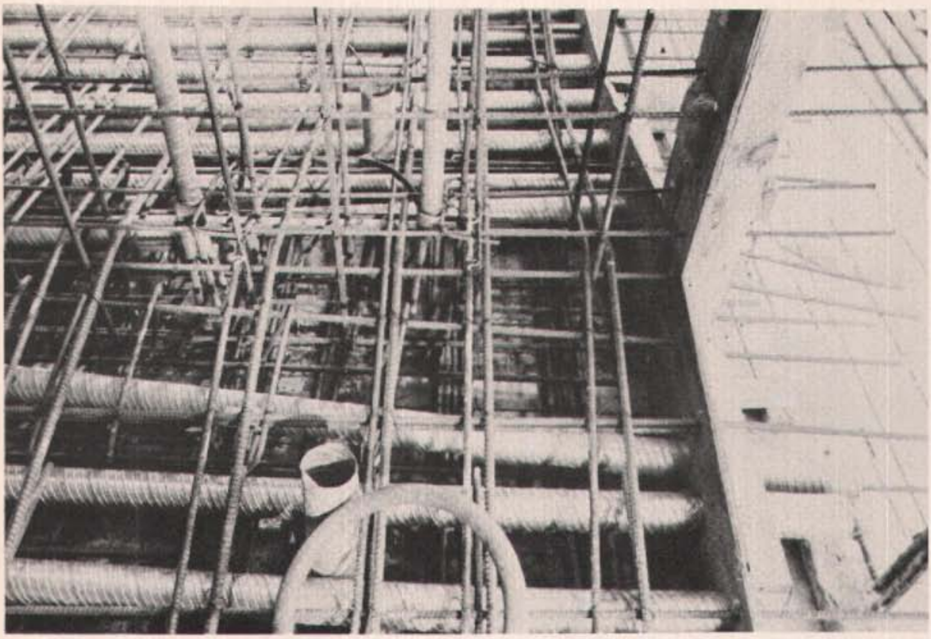


Fig. 15. Bottom slab construction.

A major effort is necessary in this type of construction concerning plan preparation. Extra care must be taken by the detailers to avoid conflicts in location in this three-dimensional structure. In addition, some thought must be given during the detailing phase to placement of concrete in the segments. For example, in the 15-ft (4.6 m) segments, no physical location was available to place a tremie hopper and chute into the center web wall.

Because of this it was necessary to temporarily cut a transverse duct in order to place a chute for concreting operations. This then required halting the concrete placement while the chute was pulled and the transverse ducts repaired. For the last ten segments the thickness of the walls was reduced to 12 in. (305 mm). This left only 5 in. (127 mm) clear between the reinforcing steel to place a chute for the concrete. This led to the use of flexible chutes that had to be cut off each time the height was adjusted.

**3. Concrete Placement.** Concreting of the segments is a major operation. The original design mix as modified by the superplasticizer met the contractor's requirement of 4000 psi (27.6 MPa) concrete in 40 hours and 6000 psi (41.3 MPa) concrete in 28 days. Because of the lost time in construction of the pier table, the contractor asked HNTB and the testing laboratory if there was any way to obtain the 4000 psi (27.6 MPa) required strength for stressing in 16 to 20 hours in lieu of the 40 hours required.

This was achieved by reducing the water-cement ratio to as low as 0.33, or approximately 29 gallons (110 litres) per cubic yard of concrete, and increasing the superplasticizer to obtain a flowable concrete. The average 28-day strength was increased to 8300 psi (57.2 MPa) from 6500 psi (44.8 MPa) and a number of cylinder breaks were in excess of 10,000 psi (68.9 MPa). Exacting requirements concerning the production of concrete were required and, at

Fig. 16. Concrete placement with crane and bucket.

the contractor's request, this function was handled by HNTB's staff in conjuction with the testing laboratory's personnel.

Placing of the concrete was difficult. The entire cross-sectional area was placed monolithically. The first placement included a 46-ft (14 m) drop in the walls as well as the floor of the box. Because of the height of the drop and the slope of the web walls, a remixing hopper was placed at the bottom of the chute which solved the segregation problem and helped produce well consolidated concrete. In accordance with an option in the specifications, HNTB required external vibration of the forms. Part of the modifications to the traveler previously discussed involved strengthening the forms to take the extra load caused by the external vibrators. External vibration combined with internal vibration through windows in the walls aided in producing well consolidated, honeycomb-free walls.

The tower crane for the project was capable of directly serving only 20 of the 58 segments to be placed (see Fig. 16). Beyond the reach of the tower crane the contractor devised a system of lifting the 2 cu yd (1.53 $m^3$) bucket to the top of the pier, unloading approximately 7 cu ft (0.2 $m^3$) to a motorized buggie, running the buggie to a ramp and dumping it into a conveyor which took the concrete into the forms. This was later modified to a forklift operation which eliminated the ramp and motorized buggie (see Fig. 17). This modification reduced the concrete placing time from 12 hours to 7 hours and greatly enhanced the quality of the concrete poured.

**4. Stressing.** The entire system of segmental construction depends on the post-tensioning system. Tension is placed in the longitudinal strands and then locked into position in order to hold the segments together. The strands themselves are composed of a very high strength steel which must be

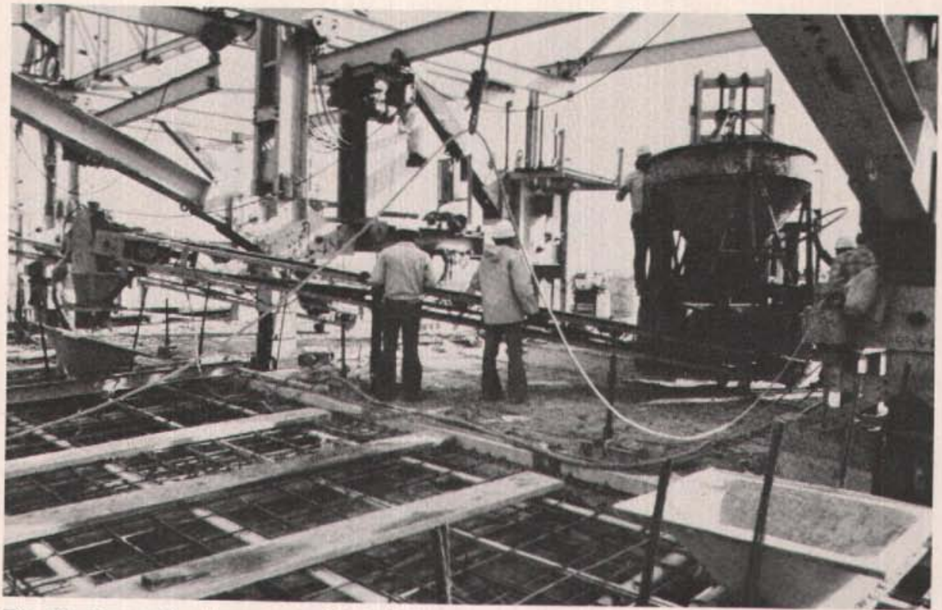


Fig. 17. Concrete placement with forklift and conveyor.

handled with extreme caution. The stressing of the 270 ksi (1860 MPa) low relaxation strands was controlled on this project by achieving the desired elongation of the strand within the limits of force allowed in the specifications as measured by the jacking pressures. In order to minimize differential stress on the structure, simultaneous stressing was required in the longitudinal tendons when stressing the large tendons. This required two-way communication between the contractor's personnel and the inspection team. Because of the amount of friction losses, dead-end stressing was a necessity. Stressing was a complex operation that required close monitoring by the engineer and a thorough knowledge of the forces and reactions involved in every phase of the stressing operation (see Fig. 18).

**5. Survey Controls.** One of the more difficult items of the segmental construction was survey control. At the start of the project concrete tests were run to determine the modulus of elasticity of the concrete and its properties in relation to creep and shrinkage. These tests required a lead time of 12 months. These concrete characteristics, together with the anticipated time of construction of the cantilever construction, were programmed into a casting curve for the project. Accurate field measurements and constant communication with the design office was maintained to provide them with updated information on how the structure was reacting.

In order to minimize the effects of differential thermal stresses in the structure, surveying was done at daybreak in the balanced condition. A joint field party of the contractor and the engineer surveyed the project with two independent sets of readings to certify the results. The use of the same instrument and the same set-up served as an excellent check.

Fig. 18. Stressing of vertical tendons.

## CREDITS

OWNER: TEXAS TURNPIKE AUTHORITY
GENERAL CONSULTANT: HOWARD NEEDLES TAMMEN & BERGENDOFF

| CONTRACT | CONTRACTOR | ENGINEER |
|---|---|---|
| South Approach Grading, Drainage and Paving<br>South Approach Low Spans<br>South Approach High Spans | Steel Construction Co.<br>Austin Bridge Company<br>Williams Bros. Constr. Co. | Turner Collie & Braden |
| Main Span | Williams Bros. Constr. Co.<br>(Prescon Corp. Superstructure Subcontractor)<br>Figg & Muller Engineers, Inc.<br>(Contractor Engineer for Design Modifications) | HNTB |
| North Approach Spans<br>Jacintoport Boulevard Bridge | Gardner B/H Contractors<br>Brown & Root, Inc. | Bernard Johnson, Inc. |
| Grading and Drainage for North Approach<br>North Approach Paving | Williams Bros. Constr. Co.<br>R. W. McKinney & T. L. James | Lockwood, Andrews & Newnam |
| Lighting<br>Administration Building<br>Fencing | Alder Electric Company<br>Howcon, Inc.<br>Universal Services, Inc. | Bovay Engineers<br>Bovay Engineers & HNTB<br>HNTB |

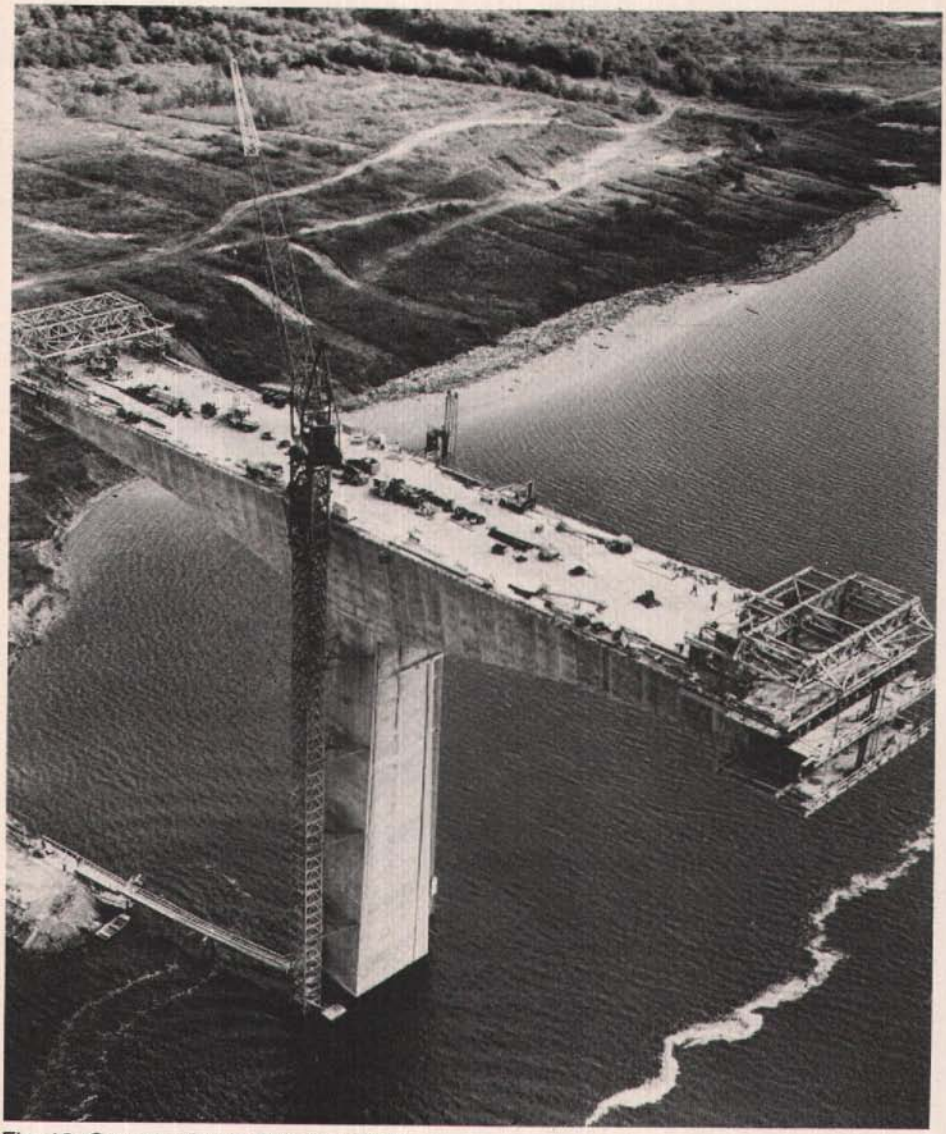


Fig. 19. Construction in October, 1981.

## SUMMARY

The Houston Ship Channel Bridge project is a 1500 ft (457.5 m) long segmentally constructed concrete bridge (see Figs. 19 to 23). Segmental bridge construction is a relatively new industry in the United States. This prestressed concrete structure is the longest of its type in North America. Being at the edge of the state of the art it was anticipated that some problems would develop during the construction of the project that would require on-site expertise as well as close cooperation and communication between the designers and field engineers. The major construction features discussed were:

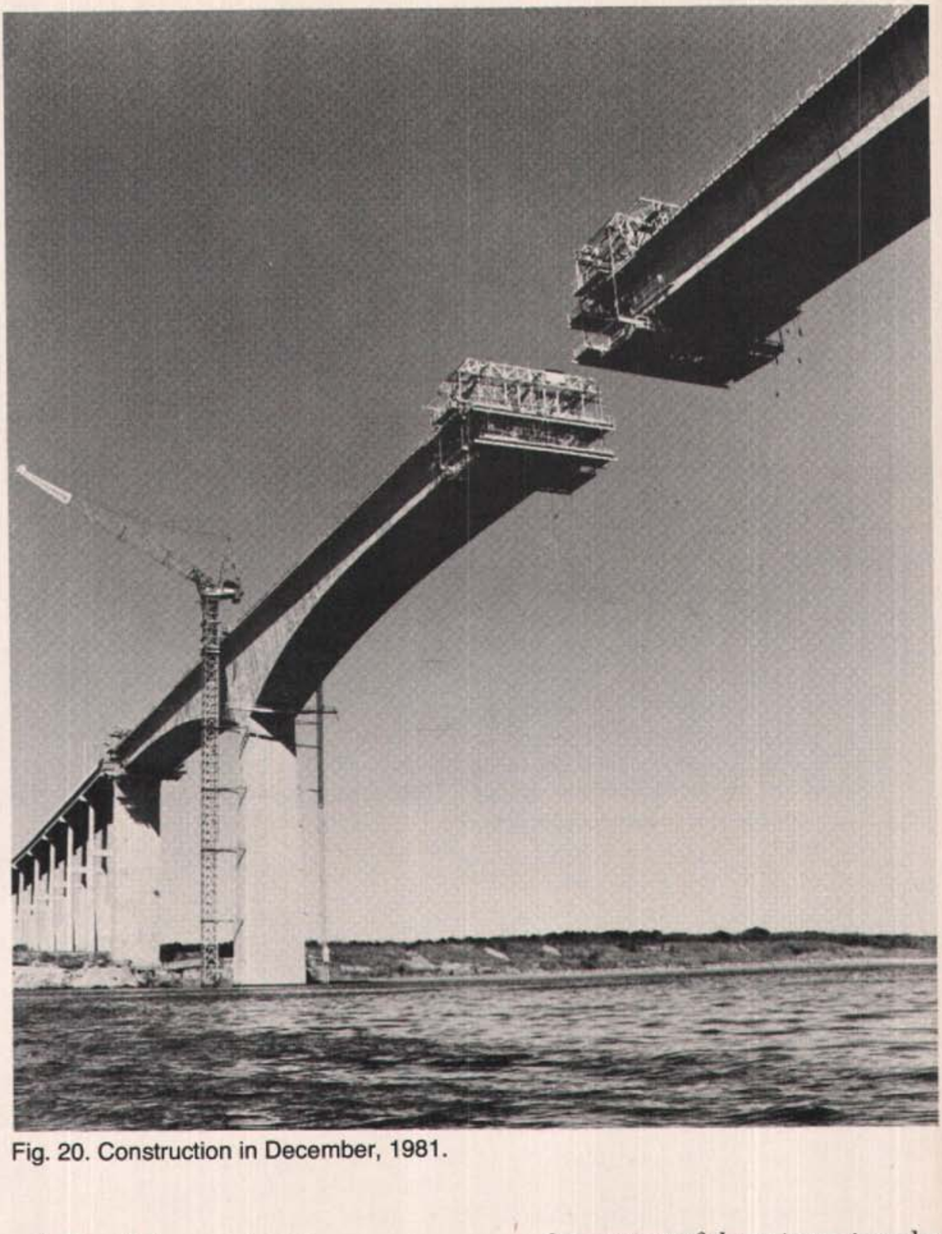


Fig. 20. Construction in December, 1981.

1. Foundation construction.
2. Thin wall concreting.
3. Problems associated with sloping web construction.
4. Problems associated with the use of high strength concrete.
5. A need for a thorough set of working drawings including a layout of embedded items using the actual dimensions of these items in order to minimize construction problems.
6. Problems associated with stressing requirements.
7. The lead time necessary for surveying controls.

The contractor realized the complexity of the project. His design modification engineer provided an on-site as-

Fig. 21. Panoramic view of completed structure.

sistant during the construction of the superstructure to facilitate decision making. The inspection team for HNTB likewise was well versed in the design and construction ramifications in this difficult type of bridge construction.

A communications channel with all concerned was opened up early which aided in keeping the problems encountered to a minimum.

The anticipated time for completion of the project was 900 calendar days. The project exceeded this time by slightly over 10 percent. The majority of the delays in construction were associated with three main factors:

1. Foundation construction problems.
2. Utilization of the high strength concrete.
3. Difficulties in placing details.

The superstructure construction exclusive of the pier tables was anticipated to take approximately 57 weeks and in actuality took 60 weeks. In order to maintain this schedule it was necessary to work 7 days a week and, in addition, provide a supporting night shift for such items as grouting, strand placement and tensioning.

The open specifications under which this project was bid is a variation of traditional bidding practices generally utilized in this country. It permitted the owner to reap the benefit of innovative design and construction practices while still retaining the owner's relationship to the design engineer of his choice. While minor problems with this concept arose, the system generally functioned well.

* * *

Fig. 22. Houston Ship Channel Bridge in use.

Fig. 23. Another view of completed structure.