Sunshine Skyway Bridge Ship Impact Design of Low Level Approaches

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Safety was in everyone's mind when a new Sunshine Skyway Bridge — an innovative cable stayed prestressed concrete structure — was authorized across Florida's lower Tampa Bay. In 1980 a main span of one of the older twin bridges had collapsed during a catastrophic ship impact. The risk of a similar impact to the new bridge was evident due to wayward vessels not only in the area of the central channel but all along the crossing, thus creating an essentially shore-to-shore ship impact risk over a 4.18 mile (6.73 km) length. During design, a decision was made that advanced today's trends toward incorporating safety, reliability, and ease of maintenance into design criteria.

That decision was to extend ship impact criteria beyond the channel spans normally protected, to include also the lengthy low level approaches — a type of structure traditionally designed for its own live and dead loads, but not ship impacts.

This made the design of the low level approaches a new type of challenge, and our firm, Parsons Brinckerhoff Quade & Douglas, Inc., was asked to take up the task. We discovered that protection could be provided, at only a modest cost increase over traditional design, by exploiting the elasticity of prestressed concrete to absorb and transfer impact loads. As a result, for apparently the first time in the United States, a major crossing (see Fig. 1) would be designed for a ship impact anywhere on its length — the first shore-to-shore protection.

This article describes the design approach and construction of the low level approaches, and the application of prestressed concrete to solving the complex problem of ship impact requirements.

Presents the design-construction method used for the low level approaches together with the application of prestressed concrete in solving the unprecedented shore-to-shore ship impact requirements of the Sunshine Skyway Bridge. The results show that prestressed concrete is an excellent material for resisting impact.

THE NEED

In 1980 the phosphate freighter Summit Venture struck the southbound (western) bridge of the Sunshine Skyway, causing a widely publicized failure and creating the need for the new Sunshine Skyway Bridge.

Before the impact, the crossing consisted of twin two-lane bridges with balanced cantilever truss main spans. The original of the pair, designed by Parsons Brinckerhoff in 1954, had been a trendsetter in its day: the most massive application yet of prestressed concrete for an American bridge. Originally a two-way bridge, it had been converted in 1971 to northbound-only to share the

Fig. 1. A finished view of the new bridge with old bridges in the background. This is the first time a major crossing has been designed for ship impact along its entire span.

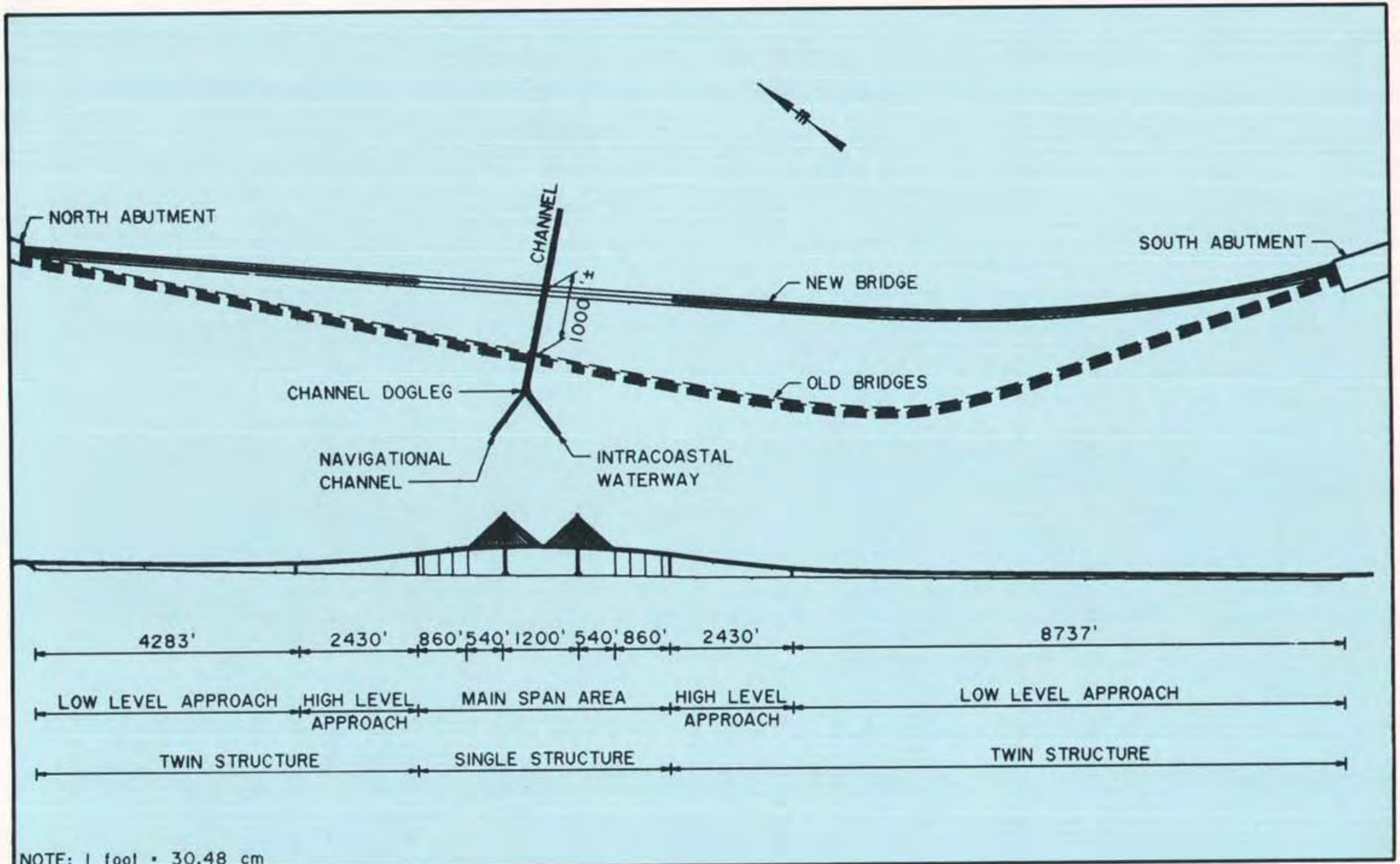


Fig. 2. Layout of new Sunshine Skyway Bridge.

traffic with its newer twin; it was this 1971 bridge that was struck by the Summit Venture, decommissioning the southbound crossing, reducing the four-lane total to two, and disrupting the local economy and commuters. The original 1954 crossing was undamaged, and again carried traffic in both directions during construction of the new cable stayed bridge. Both older bridges will eventually be demolished, with sections perhaps left (depending on the outcome of current studies) for a new life as fishing and recreational piers.

Fig. 1 shows the new cable stayed Sunshine Skyway crossing in its finished state, with the two older truss crossings in the background. Thus the new crossing, opened to traffic in mid-1988, was born not only to replace the previous crossings and improve the regional economy, but to provide an improved, safer structure — better aligned, more generous in navigational clearances, and with new criteria to resist the impact forces of any aberrant marine traffic. With its striking cable stayed main span designed by Figg and Muller Engineers and its lengthy low level approaches designed by Parsons Brinckerhoff, this new 4.18 mile (6.73 km) crossing with shore-to-shore ship impact protection is at the forefront of today's prestressed concrete bridges.

THE NEW, SAFER BRIDGE

The new bridge provided superior advantages in the form of better alignment, greater clearances and more efficient structural configuration.

Better Alignment

Safety planning began at the most basic level: where to place the bridge. There is a dogleg in the Tampa Bay Channel west of the bridge where the Intracoastal Waterway Channel meets the Tampa Bay Channel. By putting the new bridge 1000 ft (304.8 m) farther east than the old, the skew to the channel could be reduced and more space provided between bridge and dogleg. This arrangement offered better maneuverability to ship captains trying, sometimes in inclement weather, to navigate the dogleg in the channel and then get their ships properly aligned before passing between the bridge piers. This realignment supplements the impact protection built into the new design (see Fig. 2).

Greater Clearances

By nearly all measures, the new bridge is larger than either of its twin predecessors and gives ships greater clearance. For the record, some of the particulars of the new bridge are:

- Total length of crossing = 21,880 ft (4.18 miles) (6.73 km)
  - Low level approaches (twin roadways): north = 4283 ft (1.31 km) each; south = 8737 ft (2.66 km) each
  - High level approaches (twin roadways): north and south = 2430 ft (0.74 km) each
  - Main span area (single wide roadway): 4000 ft (1.22 km) (Cable stayed spans = 540-1200-540 ft) (165-366-165 m)
- Main channel width = 500 ft (152 m)
- Main span horizontal clearance = 350 ft (106.7 m) on either side of channel [versus 182 ft (55.5 m) in the old bridges]
- Vertical clearance:
  - Over the channel = 175 ft± (53.3 m)
  - At low level approaches = 20 ft ± (6.1 m)
- Water depth:
  - Low level approaches = 0 to 24 ft (0 to 7.3 m), mostly 12 to 24 ft (3.7 to 7.3 m)
  - High level approaches = 24 to 30 ft (7.3 to 9.1 m)
  - Main span area = 30 ft (9.1 m)
  - Channel = 43 ft (13.1 m)
- Tides:

Tidal range: mean = 1.3 ft (40 cm); extreme = 3.8 ft (116 cm).

Current: 0.8 to 1.0 knot (1.48 to 1.85 km per hr); 3.0 knots (5.56 km per hr) possible in outgoing tidal stream.

- Wind = 50 knots (92.66 km per hr) or more, 65 knots (120.46 km per hr) maximum recorded.

Hurricane possible every other year within 60 nautical miles (111.19 km).

Structural Configuration

From shore to shore, the new bridge complex consists of three distinct types of prestresssed concrete structures, each with appropriate considerations for impact risk assessment and protection. The dual roadway structures of the low level north and south approaches stretch from the shores, rising into high level approaches that lead in turn to the main span area. In the main span area, the parallel approach roadways merge into a single wide roadway with a central cable stayed main span and two flanking spans (see Fig. 2). These are the characteristics of the structural types:

- Low level north and south approaches. These consist of two parallel two-lane structures, one for northbound and the other for southbound. The superstructure consists of a four-span continuous reinforced concrete deck slab supported on precast prestressed concrete AASHTO Type IV girders. The substructure consists of reinforced concrete wall type piers founded on 20 in. (51 cm) square precast prestressed concrete piles. The length of each span is approximately 100 ft (30.5 m). The piers of the parallel roadways are connected across between the two structures by precast prestressed concrete frangible struts.
- High level north and south approaches. These also consist of two parallel two-lane structures, one for northbound and the other for southbound traffic. Single cell, precast post-tensioned, continuous concrete box girders, supported on concrete piers, are used. The length of each span is 135 ft (41.2 m). The foundations are supported by 24 in. (61 cm) square, precast prestressed concrete piles.
- Main span area. This consists of a single structure with a single cell, precast post-tensioned, concrete box superstructure that carries northbound and southbound traffic. The substructure consists of post-tensioned concrete piers supported by 24 in. (61 cm) square precast prestressed concrete piles. The main piers support 432 ft high (131.7 m) cable stayed pylons. The main and flanking spans are cable stayed, with a single plane of stays at the center of the two roadways.

COLLISION RISK

The risk assessment and impact criteria are now discussed.

Risk Assessment

At the initial stage of the project, the Florida Department of Transportation (FDOT), through Figg and Muller Engineers, requested COWIconsult of Denmark to perform a ship collision risk assessment study. The study is based on a mathematical risk assessment model, using available background data. The study considers the various types of marine traffic using lower Tampa Bay and identifies the main reasons for ship collisions with bridges:

- Human error (misjudgment, negligence, etc.)
- Mechanical failure (engine or steering failure)
- Environmental conditions (currents, wind, fog, etc.)

- Alignment of the bridge with respect to the entrance channel

The report stated that the designers could use probability to optimize safety and cost by distributing the risk level along the bridge structure and by accepting a tolerable level of risk. The following practical protective measures were suggested:

- Design the piers and pier shafts along the entire length for certain magnitudes of static ship impact force since superstructures themselves rarely have a substantial resistance to horizontal loads. The point of impact should correspond to the height of the hull of relevant ships.
- Raise the roadway level along the entire crossing.
- Protect the main and flanking piers with sand-filled rock islands and with dolphins.
- Shift the alignment farther east to increase the distance from the 18 degree dogleg in the navigation channel west of the old bridges.
- Consider aberrant barges as the most likely vessels to hit the bridges.

Impact Criteria

As a result of the ship collision risk assessment study, FDOT set up the following criteria for the design of the new Sunshine Skyway Bridge, in addition to AASHTO loads and their loading combinations:

- Low level approaches. The ship impact load of 1000 kips (4.45 MN) ultimate static load, combined with the dead load, should be applied at 0 to 30 degree skew and at the water level of the outer pile caps only.
- High level approaches and main span area. The following ultimate static ship impact loads, applied at 0 to 30 degree skew, and combined with the dead load should be applied:
  - Main span area = 4000 kips (17.79 MN), except 12,000 kips (53.38 MN) at the cable stayed pylon piers.
  - High level approach = 2000 kips (8.90 MN).

The ship impact load and the permanent dead loads were not to be factored. All the pier elements were to be designed for ultimate loading conditions only.

Based on the recommendations of the risk assessment study, the following improvements over the old bridge design were made:

- Increased navigational horizontal and vertical clearance of main span.
- Protection of main and flanking piers with sand islands and dolphins.

IMPACT EVALUATION

Since the risk assessment indicated that barges and other vessels might strike any portion of the bridge, the ship impact criteria had to be applied for the entire length of the bridge, including the approaches. The design team for the lengthy low level approaches, therefore, faced a perhaps unprecedented task in protecting a type of structure normally not protected against ship impact.

The remainder of this paper describes how the Parsons Brinckerhoff team arrived at the design of the ship impact protection for these low level structures. The key to our design proved to be linking the twin structures so that the force from an impact would be resisted by the piers for both roadways working together like interconnected springs.

Concepts and Alternatives

A ship impact force of 1000 kips (4.45 MN), even when it is applied at the water level, is a large force for a low

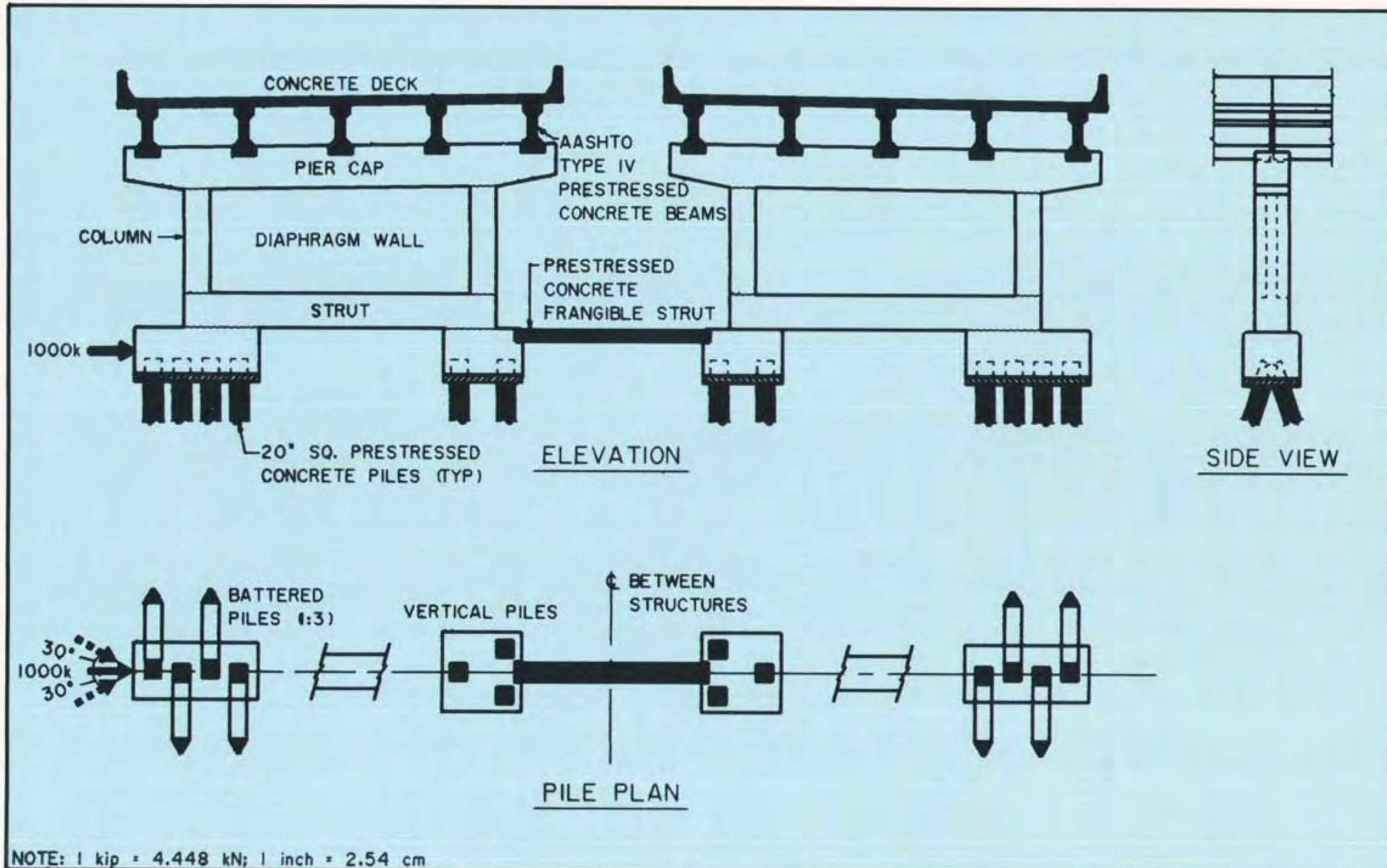


Fig. 3. Typical cross section of low level approaches.

level structure to resist. Measures such as protective nets, pile barricades, or dolphins along the entire length of such a long crossing are extremely expensive and, in many instances, impractical. Therefore, it was necessary to make an evaluation and arrive at a rational solution satisfying the stringent design criteria.

The following design criteria for the low level approaches were set prior to this evaluation:

- The previously approved pier shape had to be maintained. The pier consisted of two columns connected with a pier cap, diaphragm walls, strut above the footings, and individual footings under each column (Fig. 3).
- The superstructure consisting of AASHTO Type IV beams and a deck with no intermediate diaphragms was to be used.

The following limitations were applied to the ship impact forces:

- Ultimate load of 1000 kips (4.45 MN) was to be combined only with unfactored dead load.
- Ship impact load was to be assumed to be a static load applied at the water level.
- Ship impact load was to be applied only at the outer, not inner, pile caps (those that are exposed to an aberrant barge or ship).
- Ship impact angle was to be within ±30 degrees measured in a horizontal plane from the pier centerline (see Fig. 3).
- The superstructure was not to be designed to resist direct ship impact forces.
- It was assumed that if the ship impact force to one of the roadways (northbound or southbound) were greater than 1000 kips (4.45 MN), it would damage the impacted roadway, but the adjoining roadway would remain undamaged and open to traffic.

Geotechnical Investigation

Evaluation of a bridge's structural system for ship impact forces is influenced by the soil characteristics below the river bottom and the resulting deflection of the piles. The stiffer the soil, the smaller the deflection of the top of the pile. In the area of the low level approaches, as is usual in long approaches, the soil stratification and composition vary (Fig. 4).

In the area of the north approaches, the water depth varies from 0 to 30 ft (0 to 9.1 m), with a deep sand and shell layer around elevation −55.00 ft (−17 m). Below the sand and shell layer, clay, with intermittent silt and limestone pockets, predominates.

In the area of the south approaches, the water is shallower and varies in depth from 0 to 25 ft (0 to 7.62 m), with a thin layer of sand and shell underlain mostly with clay. Sandstone and dolomite lenses are present in the clay at a few locations.

The project's geotechnical consultants, Schmertmann and Crapps, Inc., performed an extensive geotechnical field investigation, including an elaborate field testing program. As a result of this program, the safe working load level for the 20 in. (51 cm) square prestressed concrete piles was established at 150 tons (1.33 MN), with an ultimate factor of safety of 2.25.

Vertical load on a pile was only one factor in the puzzle to design a structure for ship impact criteria. The large horizontal forces of a ship impact to any given pier also had to be accounted for. Either the piers would have to be very massive, or, conversely, they would have to be slender and flexible and so linked that a group of piers shared the ship impact force. The flexible linking of adjacent piers with a precast prestressed concrete frangible strut was the key.

To simplify the analysis and at the same time simulate the interaction of

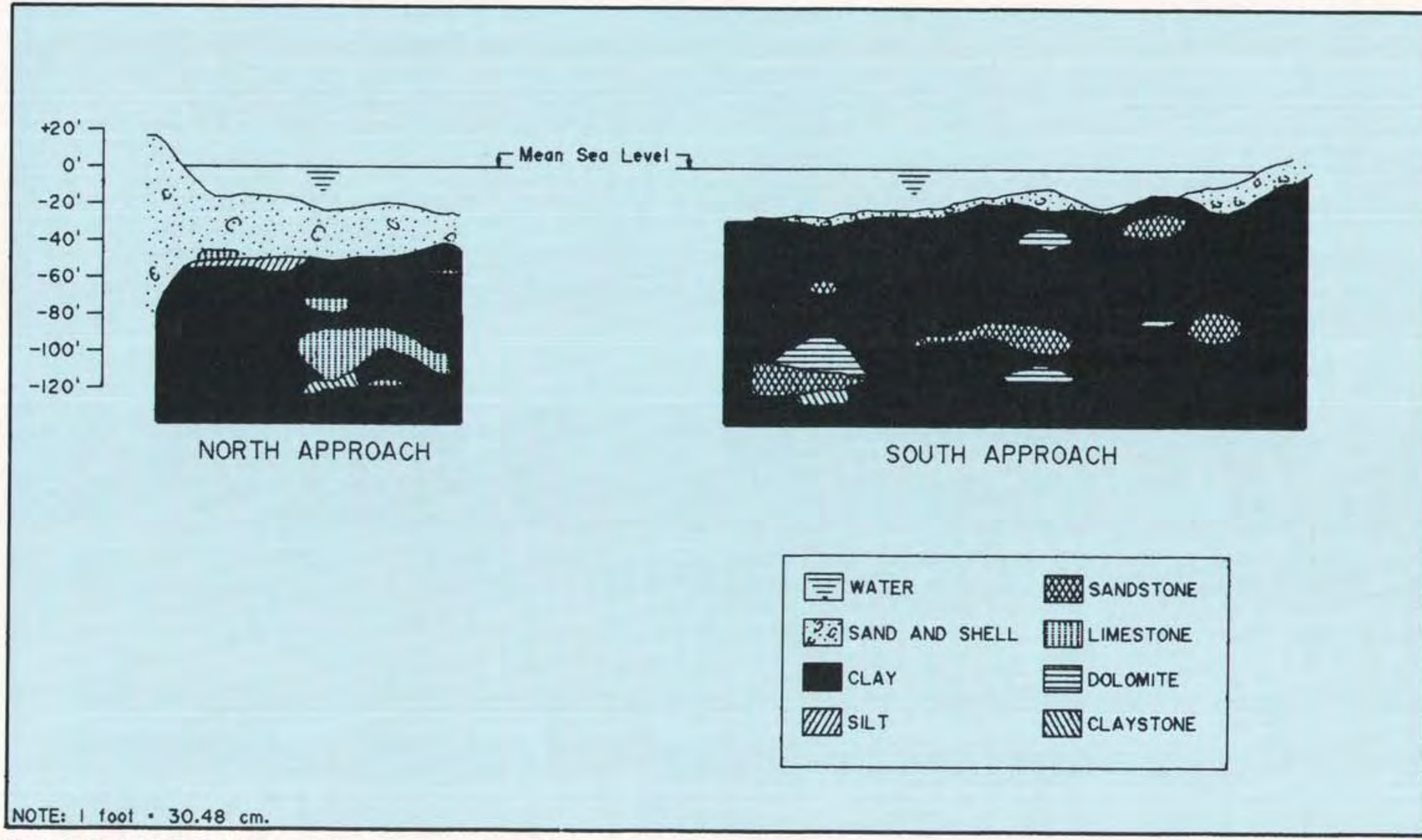


Fig. 4 Soil stratification beneath the low level approaches.

soil and structure, the "equivalent point of fixity" approach was used to model the foundation system. Using methodologies described in articles by Davisson (1965, 1970),[1,2] Reese and Matlock (1956),[3] and Penizen (1970)[4] and the finite difference computer program developed by Reese and his colleagues, the design team determined equivalent points of fixity for a unit pile expressed as a depth below the mud line beyond which the pile could be considered fixed. Equivalent points of fixity were evaluated based on an estimation of the moment curvature and deflection of a unit pile.

Equivalent points of fixity for the low level approaches of the structure can be summed up as follows:

- 15 ft (4.57 m) below mud line for piers located in less than 15 ft (4.57 m) of water.
- 12 ft (3.66 m) below mud line for piers located in greater than 15 ft (4.57 m) of water.

The above values are an average for all the low level approach piers. However, piers in the north approach had deeper points of fixity than south approach piers because of different soil conditions.

Individual pile deflections tend to be greater than a pile group's deflection. The final results of the frame analysis, using the equivalent points of fixity approach, were compared to the individual pile deflections obtained from the previously mentioned programs and were found to be less than the deflection obtained for the individual piles.

Advantages of Prestressed Concrete to Resist Ship Impact

Throughout the low level approaches, precast prestressed concrete piles supported a reinforced concrete pier. The piers supported precast prestressed concrete I-beams, and a concrete deck was poured on top (Fig. 3). In addition, a precast prestressed concrete beam was used as a frangible strut between the northbound and southbound roadway piers.

The authors believe the following advantages make prestressed concrete the suitable material to resist ship impact:

- The high load resistance of the prestressed concrete piles, both in compression and in tension.
- Ductility of the prestressed concrete piles to transmit the excess loads to adjoining foundation elements through the concrete piers.
- Ability of the stiff precast prestressed superstructure element to transmit the unresisted loads to the adjoining piers, without which, resistance to ship impact forces would have been very difficult.
- Simple transfer connection both at the foundations and at the bearing levels to transfer the high ship impact forces.
- Saving in costs due to the elimination of intermediate diaphragms because of the stiff prestressed concrete beams.
- Greater safety for the bridge not struck, since the precast prestressed concrete frangible strut can be designed to fail before transmitting too much load from the struck bridge to the adjacent one, endangering both. Thus, the strut acts as a fail-safe mechanism.

Pile Layouts

Several pile group layouts were thoroughly investigated. They ranged from six to ten piles at a pier, with the piles at several horizontal angles and also at different slopes. Most alternatives failed, due to either high compression or high tension loads in the piles. Other alternatives, though theoretically feasible, had to be discarded as impractical since they were difficult to construct and were uneconomical. With the introduction of a precast prestressed concrete frangible strut between the

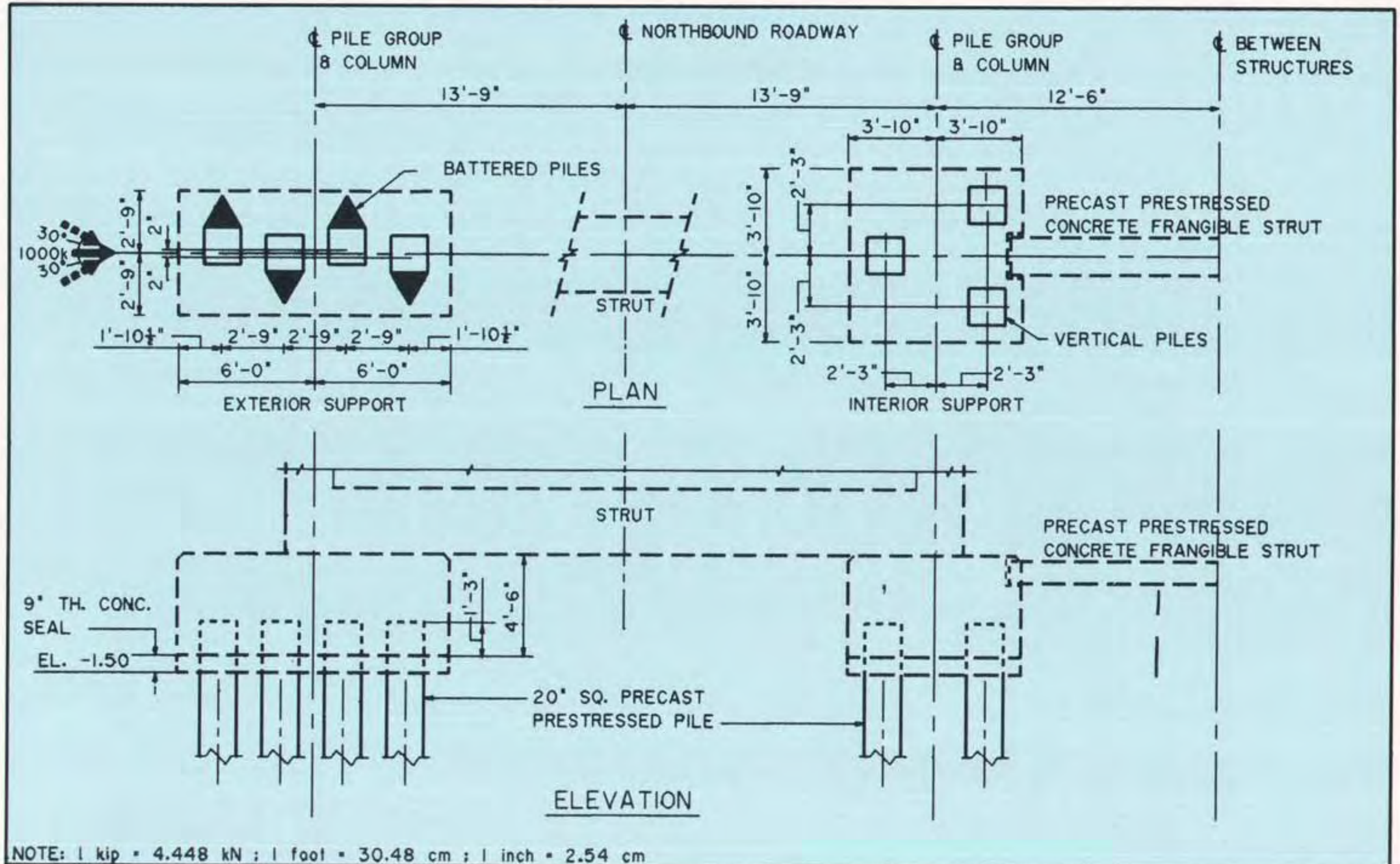


Fig. 5. The constructed foundation system.

Fig. 6. Precast prestressed concrete piles being driven using template.

adjoining structures, a practical solution was achieved.

Using the frangible strut, two workable pile layouts were developed for consideration, a six-pile system in shallow water and a seven-pile system in deeper water. The proposed six-pile system had four piles at the exterior and two piles at the interior supports, whereas the proposed seven-pile system had four piles at the exterior supports and three piles at the interior supports (see Fig. 5).

Finally, after much evaluation and discussion, a seven-pile-per-pier arrangement (in both shallow and deeper water) was selected for uniformity as well as for its increased safety in low water areas. This scheme was acceptable to the FDOT, the Federal Highway Administration (FHWA), and the contractors. The piles are shown being driven and after cut-off in photographs taken after construction was started (Figs. 6 and 7).

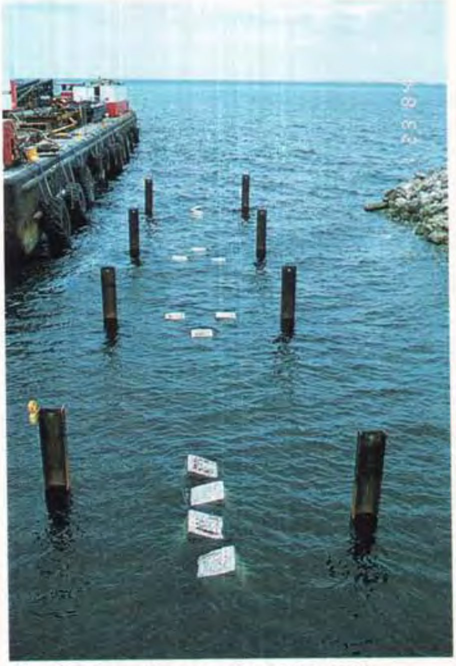


Fig. 7. View of precast prestressed piles after cut-off.

Analytical Models

Three levels of modeling were used, representing first a single pier, then a series of piers (from a single roadway structure), and finally two parallel multi-pier roadway structures.

The first level was a simplified two-dimensional model of a single pier with its foundations and did not include the superstructure. This model was used only to help in designing the larger three-dimensional models.

The second level was a three-dimensional model of a four-span continuous superstructure with the five piers of a single roadway. The individual piers were modeled as in the first-level model. No frangible strut connection was included in this second model. The results of the computer run of the second-level model clearly showed that a single structure was unable to resist the ship impact forces.

At the final third level, the second-level model was extended to include the piers and superstructure of the adjoining structure, connected through the precast prestressed concrete frangible struts (Fig. 8).

In all of the above, the piles were modeled as individual columns, fixed top and bottom. The battered piles in the exterior pile caps intersected the vertical center of gravity of the cap in order to reduce the moments in the piles. The diaphragm walls between the columns were idealized into a grid. In the second and third levels, the composite section of the superstructure was modeled as a series of line elements. The transverse continuity of the deck was simulated by the diagonal bracing elements, which helped to create the effect of a horizontal truss for transfer of loads from one pier to the next. At the bearing levels only the lateral and longitudinal moments were released. The behavior of the elastomeric bearing pads was considered as a longitudinal spring, but since the effect was minimal, it was discarded.

Fig. 9 shows the plan view of the

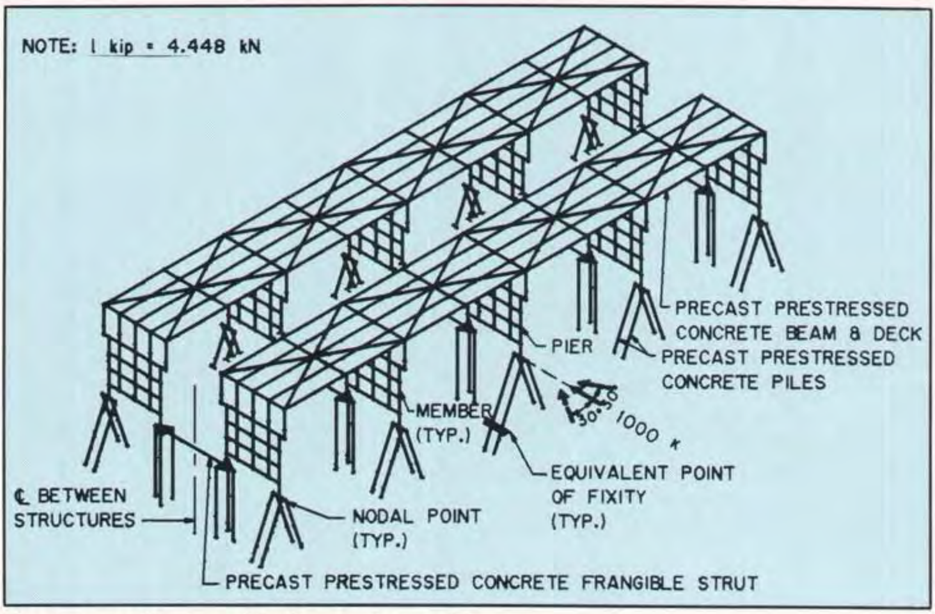


Fig. 8. Isometric view of final computer model.

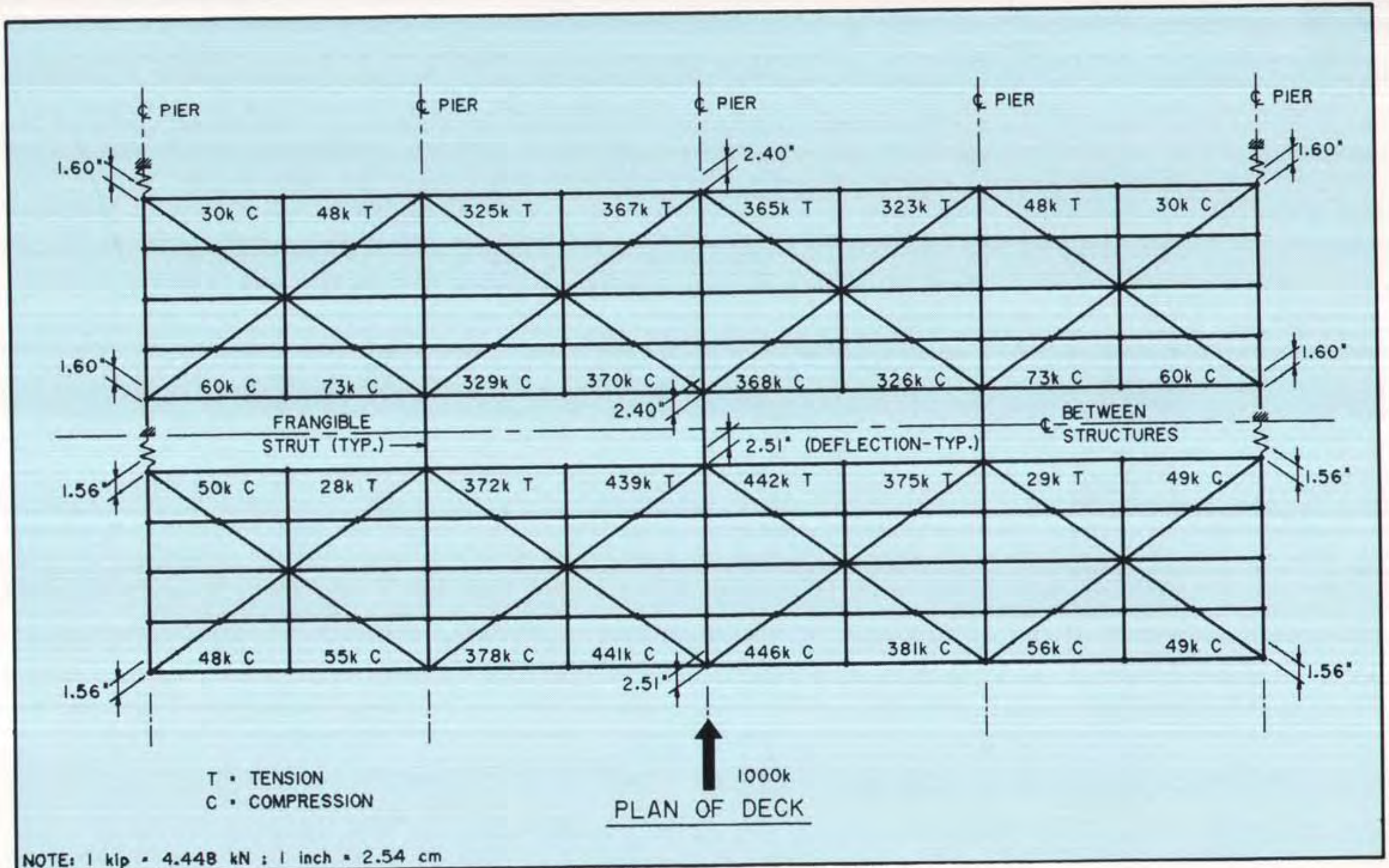


Fig. 9. Distribution of forces and deck lateral deflection from a ship impact at continuous deck pier.

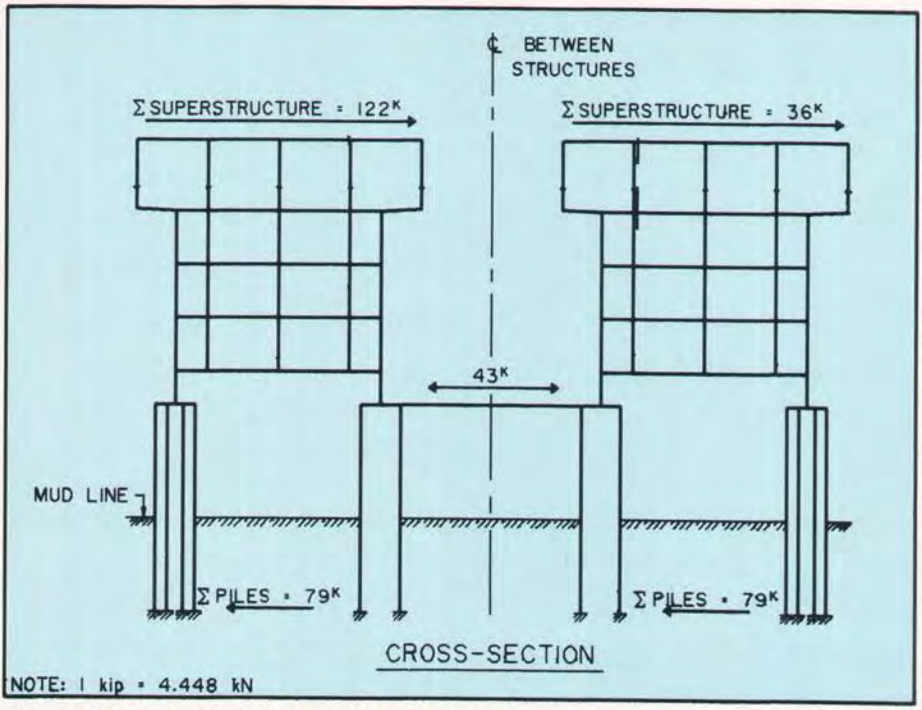


Fig. 10. Force transfer in the ship impacted pier.

model for a four-span continuous unit between deck expansion joints. The center pier was subjected to ship impact force. The lateral springs at the two ends simulated the resistance from the adjacent units through the pier shear blocks. Fig. 9 also shows the deflections and forces that resulted in the superstructure. Fig. 10 shows the force transfer in the impacted substructure elements, and Fig. 11 shows the force transfer in the adjoining substructure elements.

The model similar to that in Fig. 8 was created to simulate an expansion joint pier under ship impact. Fig. 12 shows the plan view of this model. A discontinuity was introduced in the superstructure at the expansion joint pier, and also at each end of the four-span model. Longitudinal and lateral springs were introduced at these ends to simulate a continuous deck behavior. The spring constants were evaluated utilizing additional simplified models. Fig. 12 also shows the deflections and forces in the superstructure resulting from a ship impact load at the expansion joint pier. The force transfer in the substructure is very similar to that at a continuous deck pier, as shown in Fig. 10, but with minor variations in the values.

Evaluation of the Analytical Results

A careful evaluation of the various computer outputs verified that when the ship impact angle was zero, all the piles encountered axial compression along with lateral moment, but piles of the interior pile cap experienced lower magnitude forces than piles of the exterior pile cap.

When the ship impact angle was changed laterally to ±30 degrees, the piles experienced similar but smaller forces in the lateral direction than for

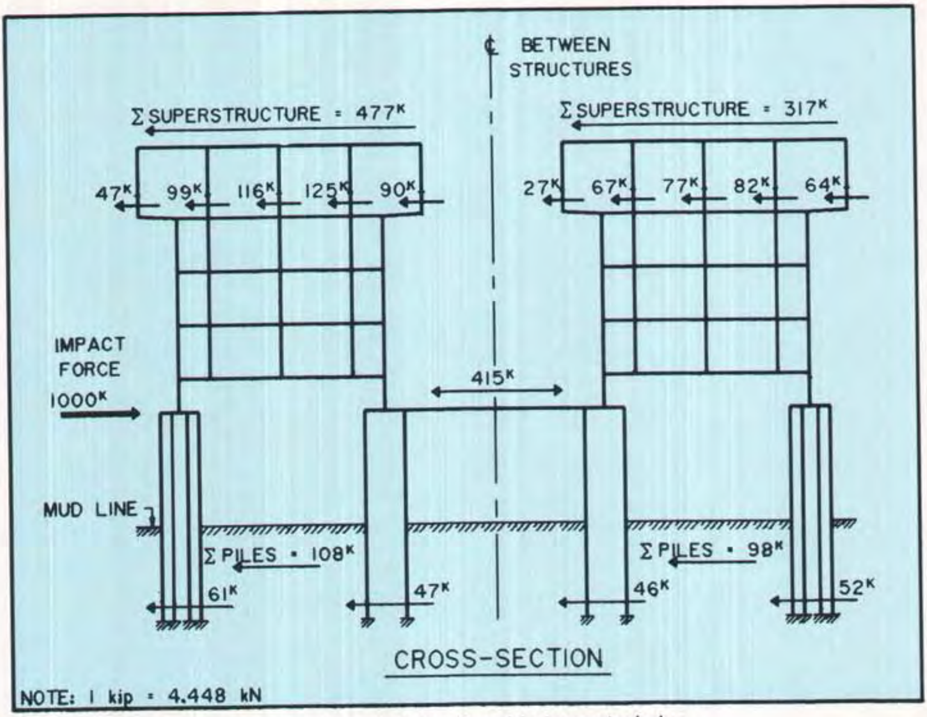


Fig. 11. Force transfer in the pier adjoining the ship impacted pier.

Table 1. Pile loads at the impacted pier.

Impact angle	Exterior pile cap piles		Interior pile cap piles	
	(Impact pier)		(Impact pier)	
	Maximum axial tension	Maximum axial compression	Maximum axial tension	Maximum axial compression
0 degrees	—	324 kips	—	181 kips
30 degrees	148 kips	619 kips	—	213 kips

Note: 1 kip = 4.448 kN.

the zero ship impact angle. However, the longitudinal component of the 30 degree ship impact force was seen to cause tension in two of the four piles of the exterior pile caps and compression in the other two. Table 1 shows the resulting pile load values for the deeper water area. The shallower water area pile forces were less critical.

As can be observed in Table 1, only the piles in the exterior pile caps underwent axial tension. However, only two piles in the group suffered axial tension, while the other two piles were subjected to axial compression. In the interior pile caps, all piles were subjected only to axial compression.

The moments were large in the lateral direction (along the axis of the pier). The longitudinal component of the ship im-

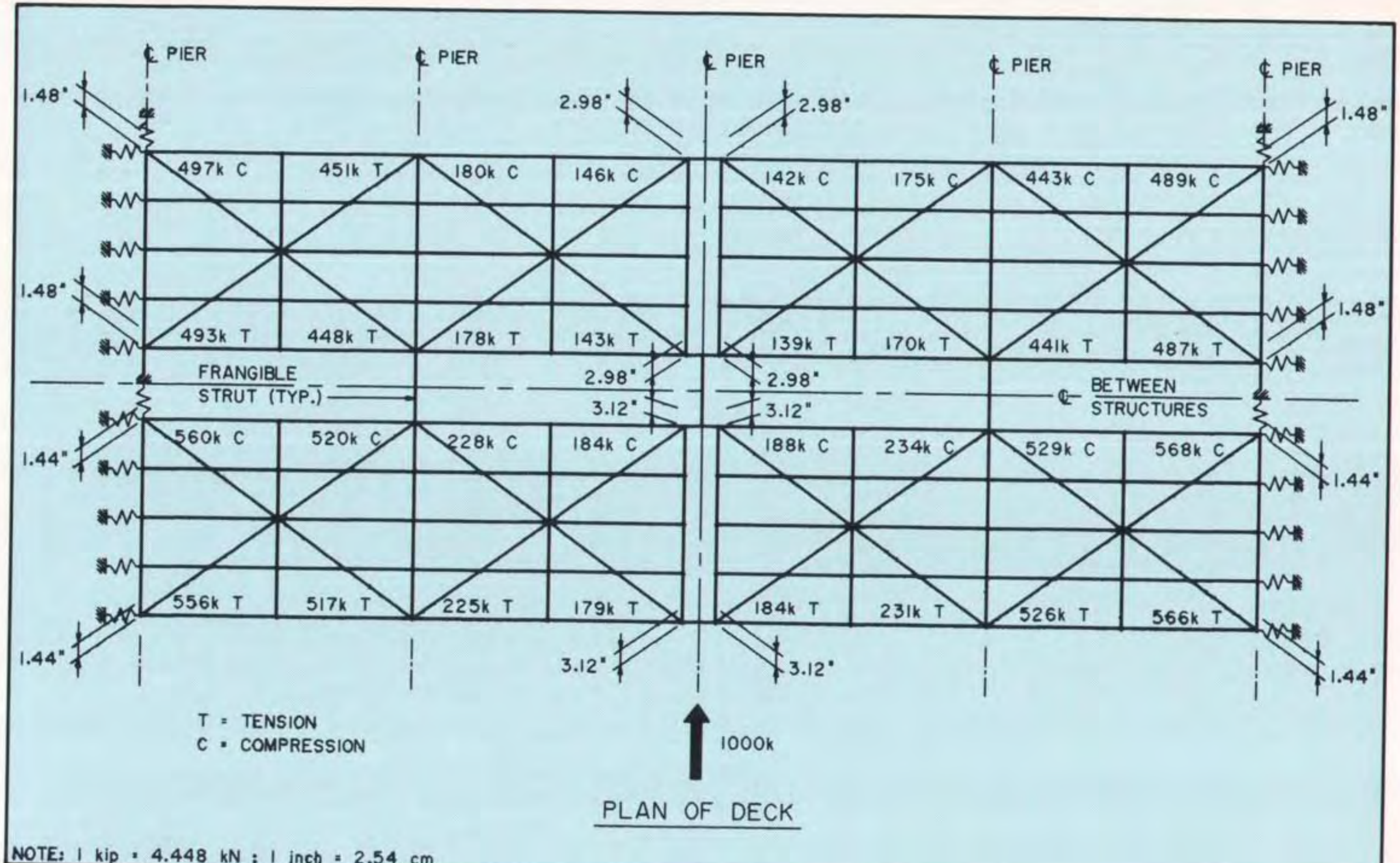


Fig. 12. Distribution of forces and deck lateral deflection from a ship impact at expansion joint pier.

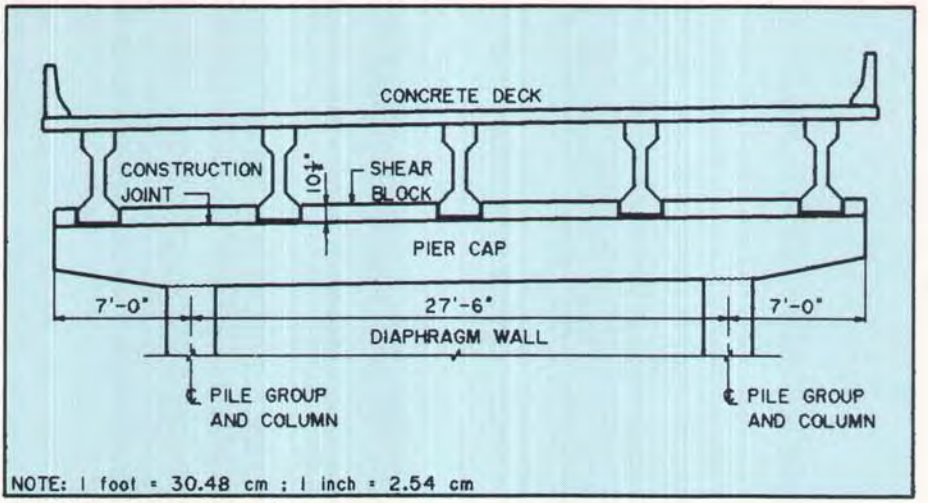


Fig. 13. Typical pier top detail.

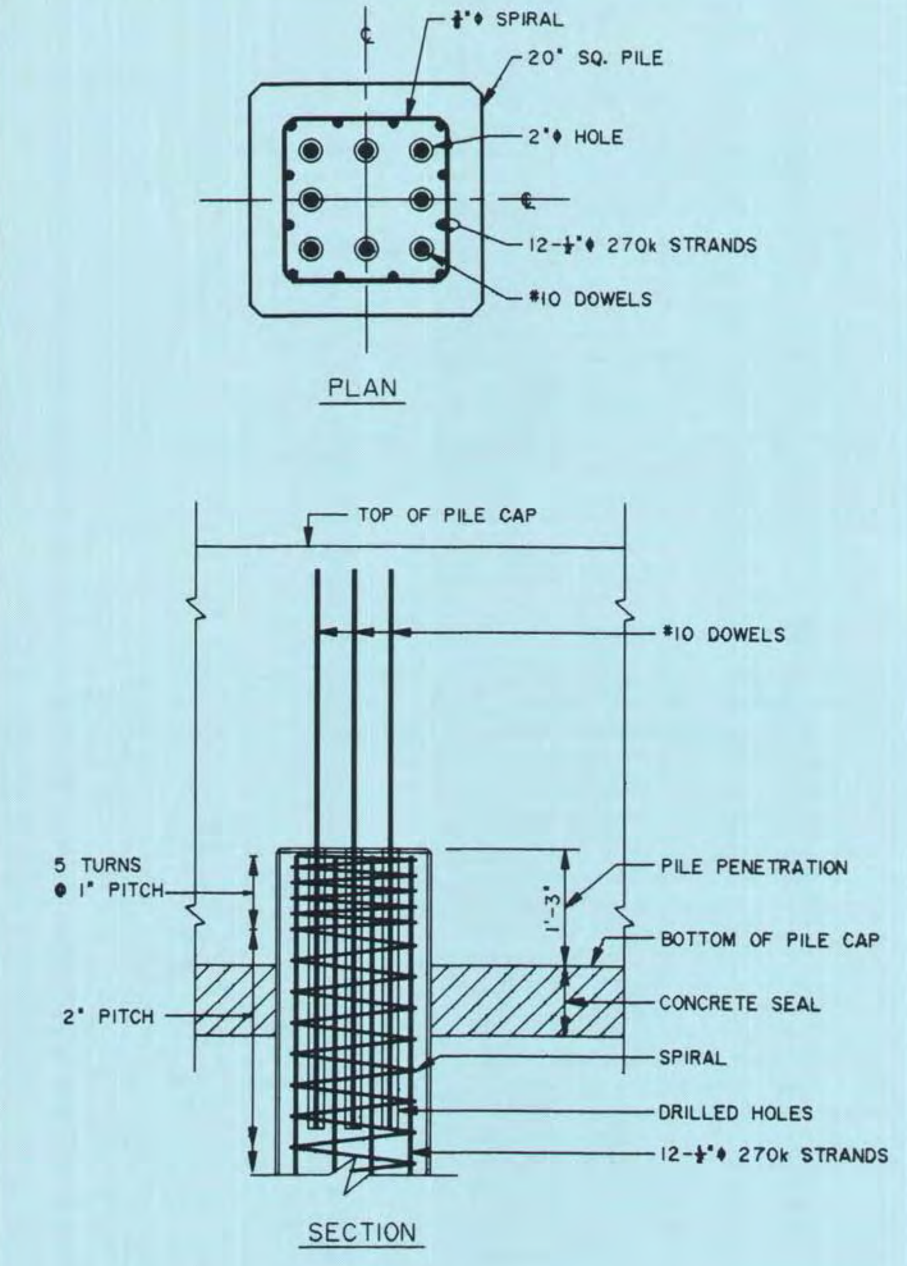
pact force was mostly resisted by the A-frame action of the piles in the exterior pile cap, and moments in the longitudinal direction (perpendicular to the axis of the pier) were minimal.

The model showed that a 420 kip (1.87 MN) force from the impacted pier, at the foundation level, would be transferred to the pile of the adjacent structure through the frangible strut. The unresisted horizontal force at the foundation level would be transferred to the superstructure elements through deflection of the piers. The force transfer from substructure to superstructure was accomplished through shear blocks at the top of the piers (Fig. 13). The combined response of the beams with the end diaphragms would transfer the force into the deck. The deck would act as a horizontal diaphragm, and by lateral bending transfer the unbalanced load into the adjoining piers. In the adjoining piers, the load transfer path would be reversed, running from the deck level to the foundations.

As a result of the various analyses, the 20 in. (51 cm) square battered piles were determined to experience maximum compression or tension loads, together with bending moments along both the principal axes. In general, buckling is critical in piles not only during high compression loads coupled with biaxial moments but also during low compression loads coupled with relatively higher biaxial moments (Hawkins 1977,[5] Nathan 1983,[6] Park and Falconer 1983,[7] Sheppard 1983[8]). In our case, the transverse moment along the pile, starting from the top, decreased gradually before reaching the point of inflection approximately at the mud line and then gradually increased up to the equivalent point of fixity at about 10 to 15 ft (3.1 to 4.6 m) below the mud line. Thus, the entire length of the pile up to the equivalent point of fixity was critical for buckling.

PROTECTIVE DESIGN

We expected the plastic hinge formation in the piles to occur just below the pile cap. In order to provide additional fixity at the pile cap level, the piles' embedment into the caps was increased to 1 ft 3 in. (38.10 cm) (Fig. 14). Drilled-in reinforcement was provided at the top of



NOTE: 1 kip = 4.448 kN ; 1 foot = 30.48 cm ; 1 inch = 2.54 cm

Fig. 14. Precast prestressed concrete pile details.

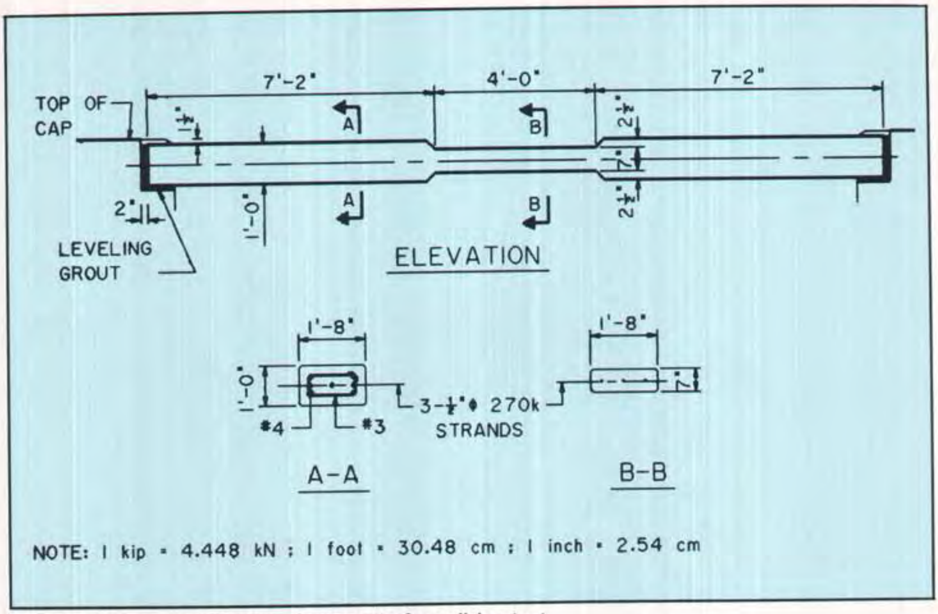


Fig. 15. Precast prestressed concrete frangible strut.

the pile and additional spiral reinforcement was provided in the upper 10 ft (3.1 m) of the pile to increase ductility and reduce any damage due to plastic hinge formation. We expected also that the plastic hinge formation would occur only in the battered piles of the exterior pile caps. In our opinion, at the onset of plastic hinge formation, redistribution of forces between the piles in the exterior pile caps and also to a certain extent with the other piles along the entire cross section would take place, reducing the damage. Any such damage inflicted on the battered piles of the exterior pile caps would be minor and could be repaired.

The precast prestressed concrete frangible strut connecting the two adjoining piers was carefully designed for a critical buckling load (Michalos and Wilson 1971[9]) of 450 kips (2.00 MN) [slightly higher than the 420 kips (1.87 MN) transfer force]. The strut carries a minimum amount of prestress force to eliminate hairline cracking due to its own dead load deflection. Since the critical buckling load is theoretical, and can vary due to actual in-place strength of concrete, elastic modulus, etc., it was decided to test a few of the struts with minor variations to optimize their cross section. The successful strut shape was then finalized (Fig. 15).

All the various pier elements such as the strut above the footings, columns, diaphragm wall, pier cap, and shear blocks were carefully designed to resist the ship impact transfer force. Strengthening of the end diaphragms and additional reinforcement at the top and bottom of the deck slab were required to transmit the horizontal forces. Fig. 13 shows the details at the top of a pier and Fig. 16 shows a typical continuity diaphragm and deck detail.

At the expansion joint pier, an additional element of horizontal restraint was provided, and the end diaphragm was strengthened to resist the longitudinal tension force (Fig. 17). The longitudinal restrainer system was designed to accommodate the normal thermal movements of the structure.

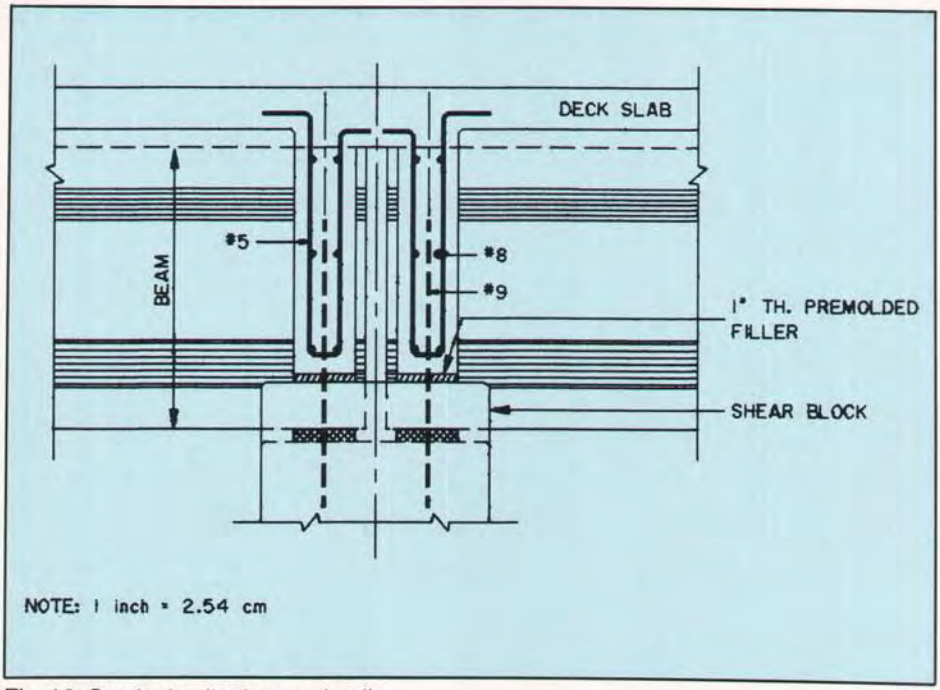


Fig. 16. Continuity diaphragm detail.

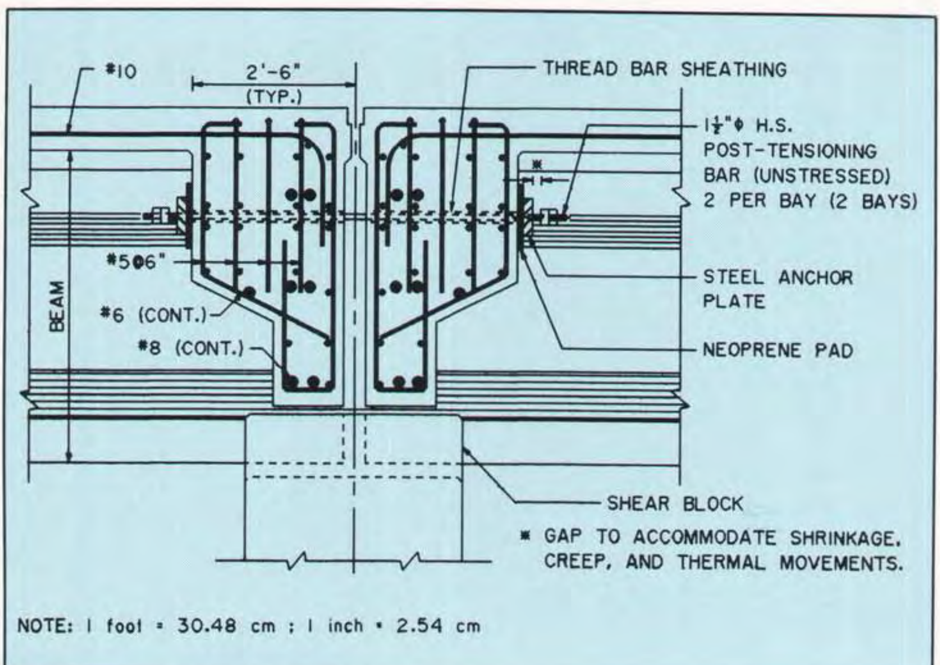


Fig. 17. Expansion joint diaphragm detail.

Fig. 18. Pile caps after construction.

CONSTRUCTION OF APPROACHES

Construction of the piles and of the beams and deck are now described.

Piles

The construction of the low level approaches was started in late 1983 and finished by late 1986. The precast prestressed piles for each pier were driven in position using a template (as shown in Fig. 6). The batter piles and the vertical piles were then cut perpendicular to their axes (Fig. 7) prior to pouring the pile caps (Fig. 18). The strut and pier construction then followed (Figs. 19 and 20). Careful attention was paid to prefabricating reinforcing bar cages, off-site, so that they could be slipped into their respective forms. This made the construction of the pier units easier and more economical since minimal tying of reinforcing steel was necessary in the field. During the pier construction, as is usual, some piles were out of place after they were driven.

This was corrected by either adding additional piles to the bent if the error was too much; or, if the error was minor, the bent was reevaluated, and in most instances construction was allowed to proceed. However, a continuous communication channel was open during construction between the designer and the field personnel in order to cut delays when problems occurred.

Beams and Deck

After a few piers were constructed, the erection of the prestressed concrete beams followed (Fig. 21). At the same time, additional piles were being driven and piers constructed farther up the line. Fig. 22 shows a continuity diaphragm after construction.

The reinforced concrete deck was poured after a few spans were erected

Fig. 19. Pier under construction. Strut on top of pile caps in the background.


Fig. 20. Piers after construction.

Fig. 21. Precast prestressed concrete beams after erection.

(Fig. 23). The continuous deck slab was poured as one unit from expansion joint to expansion joint, starting from one end and proceeding to the other. Fig. 24 shows the precast prestressed concrete frangible strut in place between the piers.

Fig. 25 shows the completed view of the Sunshine Skyway Bridge.

During construction, close contact was kept between the designers and field personnel. All field problems were solved within a short period of time and the structure was successfully completed.

COSTS OF MEETING IMPACT CRITERIA

The need to meet the ship impact criteria does increase the cost of a structure, both during design and during construction. The design cost increase is

Fig. 22. Deck continuity diaphragm.

Fig. 23. Bird's eye view of deck slabs after pour.

Fig. 24. Precast prestressed concrete frangible strut in place.

Fig. 25. Completed view of Sunshine Skyway Bridge.

difficult to quantify, but it is minimal. The construction cost can be more readily estimated. The ship impact criteria required the following additional items:

- Additional precast prestressed concrete piles.
- Increased reinforcement in the strut between the pile caps.
- Introduction of shear blocks at the tops of the piers.
- Additional reinforcement at both faces of the deck to resist longitudinal bending (in plan) of the deck.
- Introduction of precast prestressed concrete frangible struts between adjoining roadways.
- A complicated interconnection of the expansion joint diaphragms.

The above items added about 15 to 20 percent to the construction cost of the project when compared to a structure designed conventionally without ship impact criteria.

CONCLUDING REMARKS

The stringent criteria for the design of the low level approaches necessitated evaluating several configurations before arriving at a practical engineering solution. Several alternative solutions were feasible in principle but involved difficult construction. The results, presented in this paper, were thoroughly scrutinized and were found satisfactory by FHWA and FDOT. Several discussions preceded the final decision.

For the designers of future prestressed concrete crossings over navigable waterways, the Sunshine Skyway project suggests that ship impact protection can effectively be extended to the entire crossing, including low level approaches. The elasticity of prestressed concrete enables the designer to meet impact criteria without massive pier structures or costly protective structures surrounding the piers. Designers using the Sunshine Skyway experience as a model should bear in mind that it was a pioneering effort; the designers of the low level approaches were limited to those solutions that were compatible with the basic design concepts already established.

For the Sunshine Skyway Bridge — a long structure in an area where aberrant marine traffic is frequent — ship impact resistance was a critical design requirement. Impact resistance was successfully achieved in the design by utilizing the elastic and ductile properties of the various elements and letting the structure deflect and thus relieve the energy due to any possible ship impact. Though the design was complex and many trial and error procedures were necessary, still a structure was designed and successfully built for the ship impact criteria. The increase in cost over a conventional design is small when compared to the protection attained and the reduction in potential disruption to the area's commerce.

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