NBSIR 84-2892

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NATIONAL BUREAU OF STANDARDS

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U.S. DEPARTMENT OF COMMERCE, Malcolm Baldrige, ***Secretary***
NATIONAL BUREAU OF STANDARDS, Ernest Ambler, ***Director***

ABSTRACT

The General Accounting Office (GAO) requested the assistance of the National Bureau of Standards in the investigation of the construction of the new Sunshine Skyway Bridge in Florida. Specifically, GAO desired answers to questions related to the following: 1) the formation of cracks in the main piers of the bridge span; 2) the materials used in the concrete mixtures; and 3) the procedures used in the placement of concrete in the drilled shaft foundations. The objective of the GAO inquiry is to determine the reasonableness and validity of the positions taken by the Florida Department of Transportation on each of the concerns expressed by a number of individuals in connection with the bridge construction. This report provides answers to the questions and provides explanations for each answer.

It is concluded that the cracks in the piers will not affect the structural capacity of the piers. However, the cracks should be sealed as a precaution against the intrusion of additional seawater, which may cause localized corrosion of the reinforcement at any existing breaks in the epoxy coating on the reinforcing steel. The coarse aggregates that were used appear to have resulted in a good quality concrete which should be durable. Finally, there is no basis for concern with regard to the integrity of the drilled shafts.

Keywords: Bridges; concrete; construction; cracking; investigation

TABLE OF CONTENTS

1.0 INTRODUCTION

On April 11, 1984, the National Bureau of Standards (NBS) received a request from Florida Congressmen C. W. Bill Young, Sam Gibbons and Andy Ireland to provide technical assistance to investigators of the General Accounting Office (GAO) in their review of the construction of the new Sunshine Skyway Bridge. On April 16, 1984, a representative from GAO met with NBS structural engineers to discuss the nature of the GAO investigation and to present copies of technical documents that had been gathered by GAO. In addition, GAO presented a list of specific questions related to the construction of the Sunshine Skyway Bridge. The NBS engineers performed a preliminary review of the documents and questions. On May 1, 1984, NBS responded to the Congressmen's request and agreed to assist the GAO in its investigation.

The underlying objective of GAO's questions is to determine the reasonableness and validity of the position taken by the Florida Department of Transportation (FDOT) on the concerns raised by the Congressmen. This report presents the responses to the questions and explanations for the answers. The responses are based on the study of documents provided by GAO plus other technical references related to the questions. Additional information was obtained in a site visit during May 16-18, 1984.

The GAO made similar requests for assistance to the U.S. Corps of Engineers and the Bureau of Reclamation. During the course of this work, NBS maintained contact with the Corps of Engineers and the Bureau of Reclamation. The concerns and the GAO questions were discussed, in general terms, among the representatives of the three agencies. Responses to the questions, however, were independently prepared by each agency. Draft copies of these responses were distributed among the agency representatives for information purposes. The responses provided in this report represent the views of the National Bureau of Standards.

The following chapter provides background information to assist the reader who is unfamiliar with the circumstances leading up to the GAO investigation.

2.0 BACKGROUND

The new Sunshine Skyway Bridge is a 21,878 ft long structure being built at the mouth of Tampa Bay to replace the existing bridge connecting St. Petersburg with Bradenton, Florida. The twin-bridge structure being replaced originally consisted of a northbound span and a southbound span. On May 9, 1980, a tanker collided with the southbound span and a portion of the bridge fell into the bay. The collapsed portion was never repaired and the northbound span continues to carry traffic in both directions. A decision was made to build a modern replacement structure, and in 1982 construction commenced for the new Sunshine Skyway Bridge.

The new Sunshine Skyway features a 3-span, cable-stayed bridge providing 175 ft. of vertical clearance over the shipping channel. As indicated in Fig. 1, the main span will be 1200 ft. long with 540-ft. side spans. The bridge will be built using the technique of precast segmental construction, in which precast concrete box girder segments are lifted into place and made continuous by post-tensioning.

The Sunshine Skyway Bridge is being built under three separate contracts:

- -Contract 1 to construct the main pier foundations for the cable-stayed bridge,
- -Contract 2 to construct the high level approach spans and the cable-stayed bridge, and
- -Contract 3 to construct the low level approach spans (called the trestle portion).

The questions raised by GAO deal with the construction covered by Contract 1.

The cable-stayed bridge has two towers to support the single plane of stay cables. Below the roadway, the main supports consist of dual elliptical columns resting on reinforced concrete piers. Each pier is supported on reinforced concrete drilled shafts. Figures 2 and 3 show the cross sections of the piers. The piers are composed of three structural elements:

1) the top slab,
2) the hollow cone section, and
3) the bottom ring.

The top slab carries the load from the columns to the hollow cone, and the cone transfers loads to the bottom ring. The ring transfers the load to the drilled shafts.

Figure 3 shows the dates of concrete placement in the various portions of the north and south piers (the bridge runs approximately in the north-south direction). Shortly after the cone of the north pier was placed, a representative of FDOT detected cracks in the cone. A representative of the designer inspected the cracks and informed FDOT that the cracks would not impair the structural capacity of the piers. The designer also informed FDOT that the cones should be filled with water.

On October 20, 1983, a FDOT diver submitted a report to FDOT on his inspection of the exterior and interior of the piers. The report noted that cracks existed in both cones, some of which extended the full cone height of 14 ft.

In addition, it was reported that some of the cracks had efflorescence stains and "light corrosion stains".

In early November, 1983, a local television station reported on the existence of cracks within the cones, and widespread media coverage of the situation ensued. On November 11, 1983 Congressman Young dove into one of the hollow cones to personally inspect the cracking. After the inspection, Young stated that an investigation should be carried out to explain the reason for the cracks. On November 12, FDOT decided to begin pumping water out of the cones to permit more detailed inspection of the cracks and the associated efflorescence deposits.

On November 17, 1983, Congressmen Young, Gibbons, and Ireland submitted a request to GAO to initiate an investigation into the construction of the new Sunshine Skyway Bridge. On April 11, 1984, the Congressmen requested that NBS provide technical assistance to GAO in answering their concerns about the cracks in the cones, the materials used in the concrete, and the procedures used in placing the drilled shafts. The same request was also submitted to the Bureau of Reclamation and to the Corps of Engineers.

A representative from GAO met with NBS engineers to discuss GAO's technical questions. Documents relating to the technical concerns, which were obtained from FDOT, were given to the NBS engineers. These documents, along with others subsequently supplied by FDOT, were used to help formulate the answers to the questions. The following is a list of the types of documents that were provided:

- Inspection reports on the cracks in the cones
- Structural drawings and project specifications
- Concrete mixture proportions
- Results of laboratory tests on concrete mixtures
- Placement records and cylinder strength results
- Report of the concrete consultants
- Excerpts from the Massachusetts Institute of Technology (MIT) report dealing with the Sunshine Skyway
- Proposals for instrumentation of the bridge
- Copies of media coverage of the construction

Additional information was obtained from a site visit during May 16-18, 1984. The visit included an inspection of the interior of the cones and meetings with FDOT personnel. Representatives from the Bureau of Reclamation and the Corps of Engineers also participated in the site visit. The representatives from the three agencies cooperated in all aspects of the site visit, and on May 18, they held a meeting to discuss, in general terms, their proposed responses to GAO's questions.

In total, GAO posed 14 questions. The questions related to three major concerns:

1) the significance of the cracks in the cones of the piers,
2) the materials used in the concrete, and
3) the procedures used in placing the drilled shafts.

The following chapter provides the answers and explanations for the answers to the questions.

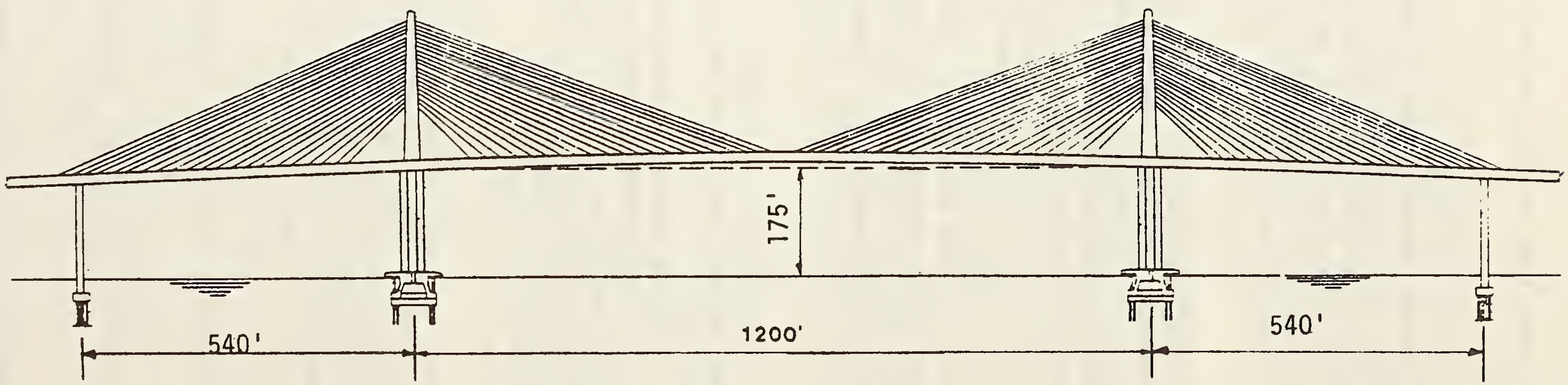


Figure 1 Elevation view of cable-stayed portion of new Sunshine Skyway

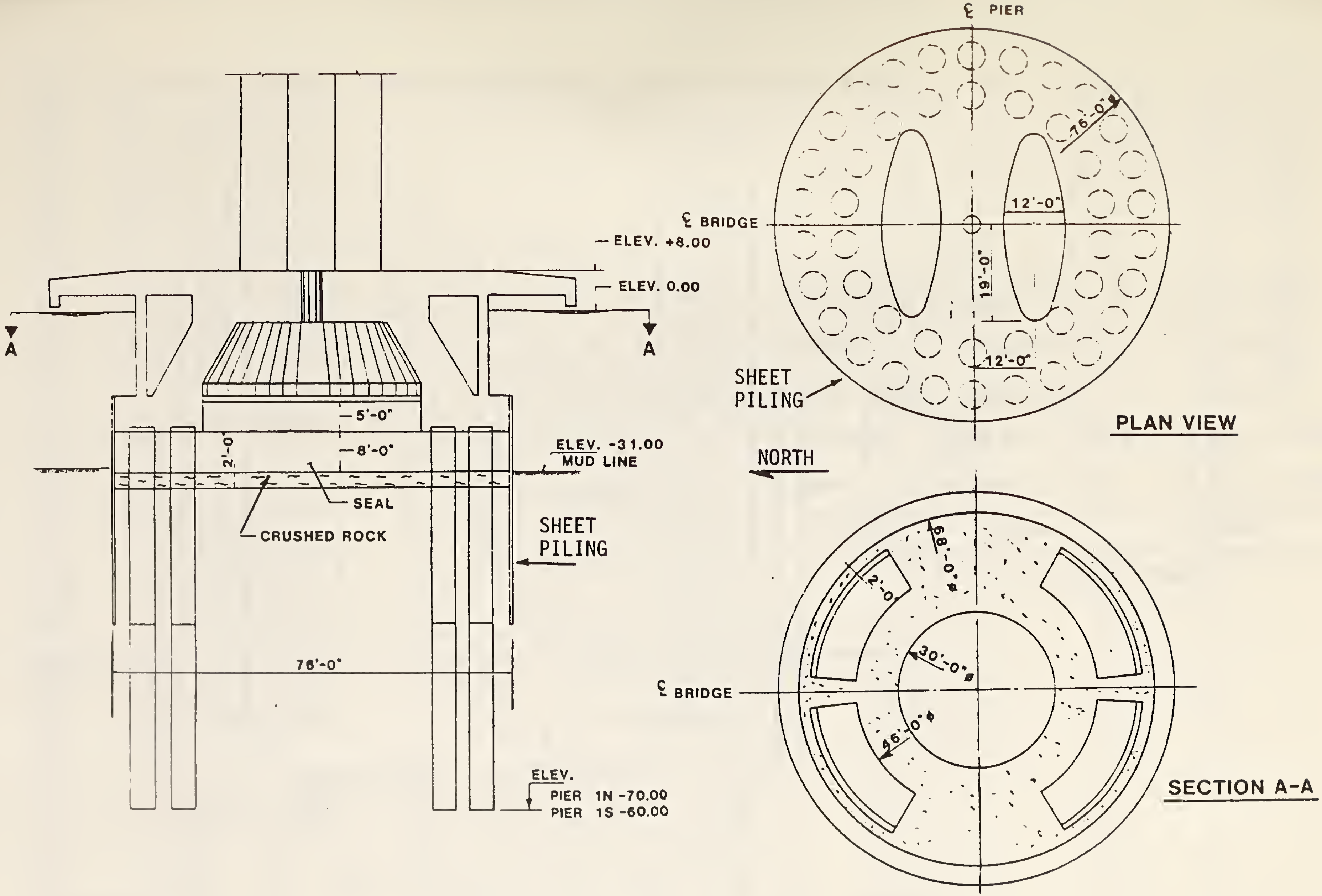
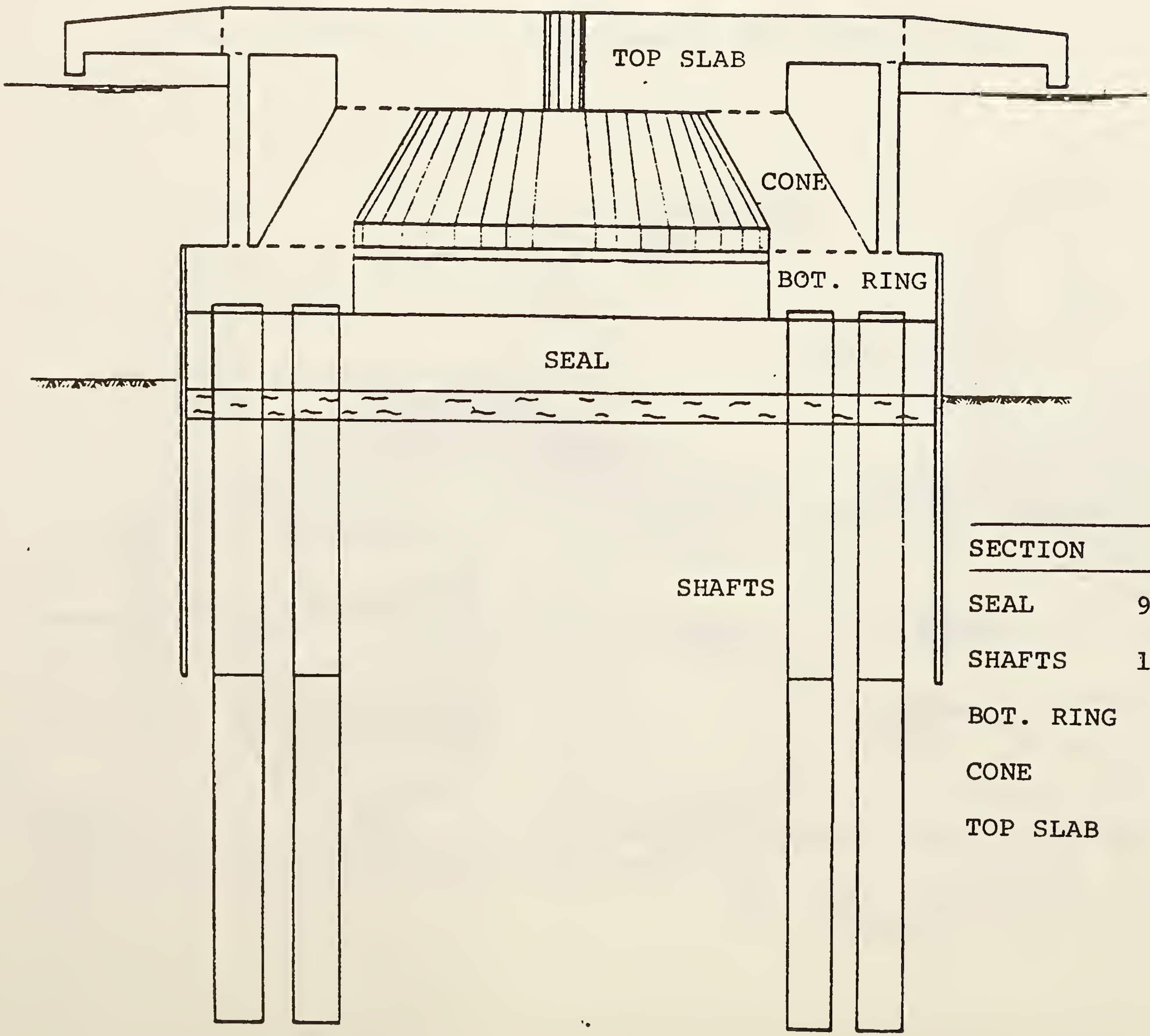


Figure 2 Cross sections of main piers for cable-stayed bridge



PLACEMENT DATES

SECTION	NORTH	SOUTH
SEAL	9/24-9/25/82	2/24-2/25/83
SHAFTS	11/12-12/30/82	2/11-3/31/83
BOT. RING	3/23/83	5/19/83
CONE	5/27/83	7/1/83
TOP SLAB	7/19/83	8/10/83

Figure 3 Dates of concrete placement in various sections of piers

3.0 RESPONSES TO QUESTIONS

QUESTION 1.1
In mass concrete pours are cracks, such as those described in the packet provided you, to be expected as normal shrinkage cracks?

RESPONSE
Yes, because in mass concrete it is difficult to prevent the formation of "shrinkage cracks". Efforts are usually taken to control the spacing and width of the cracks that form.

EXPLANATION
"Shrinkage cracks" result when concrete attempts to decrease in volume but is restrained from doing so. Because of the restraint, tensile strains (and stresses) are induced in the concrete. When the induced strains exceed the strain capacity of the concrete, cracks develop and extend into the concrete. Crack extension ceases where the induced strains are less than the strain capacity. To prevent the formation of these cracks it is necessary to control the induced tensile strains.

The following is a brief explanation of the probable source of the induced tensile strains in the cone sections of the piers. Concrete gains its strength from chemical reactions between the cement and the water. These chemical reactions give off heat and the heat of hydration of the cement is a quantitative measure of the total heat evolved up to a given age. For example, a heat of hydration of 70 cal/gram at 7 days means that each gram of cement will have evolved a total of 70 calories of heat after reacting with water for seven days. The heat of hydration will cause the temperature of the concrete to rise. The maximum temperature attained by a given structural member will depend on many factors which are discussed in detail in Ref. (1). One of the important factors is how readily the heat of hydration is dissipated through the member into its surroundings. A member having a large ratio of surface area to volume can dissipate the heat of hydration more readily than a member with a small ratio. The temperature rise would therefore be higher for the member with the low surface to volume ratio. The term "mass concrete" refers to members whose dimensions are such "that measures should be taken to cope with the generation of heat and attendant volume change to minimize cracking (1)." Note that this definition says "minimize" not "prevent" cracking.

The initial temperature rise of the concrete causes it to expand, and if the expansion is restrained compressive stresses are induced. However, at early ages the stiffness (Young's modulus of elasticity) of the "...concrete is so small that compressive stresses induced by the rise in temperature are insignificant even in zones of restraint and can be ignored (1)." After a concrete member reaches its peak temperature, it begins to cool, causing a tendency for contraction. "If a concrete member has a uniform tendency to contract but is restrained at its base or at an edge, cracking will initiate at the base or restrained edge where the restraint is greatest and will progress upward or outward until a point is reached where the stress is insufficient to continue the crack (1)."

During the construction of the piers, concrete was placed in the cone sections after the bottom rings were built. For the north pier the cone was placed 65 days after the bottom ring, and for the south pier the age difference was 43 days. When the cone concrete for each pier started to cool the bottom rings prevented the cones from freely contracting. As a result, tensile strains (and stresses) were induced in the cones, and when these strains surpassed the capacity of the concrete, cracks developed. As indicated in the above quote from Ref. (1), the cracks would be expected to start at the bottom of the cone wall and propagate toward the top. Ref (1) further states:

> "...unreinforced concrete walls or slabs, subject to base restraint, will ultimately attain cracks through the full block height spaced in the neighborhood of 1.0 to 2.0 times the height of the block."

According to the cracking survey supplied by FDOT, the average lateral spacing between the vertical cracks in the cone of the north pier was approximately 13 ft. and for the south pier the average was 11 ft. The vertical height of the cone walls is 14 ft. Thus the average crack spacing is in agreement with the above statement.

Note that the above quotation concerning crack spacing refers to unreinforced concrete. In reinforced concrete members, reinforcing steel is usually provided to control the width of "shrinkage cracks". In the case of the cones, page 9 of the structural drawings indicates that about 50 #8 bars are placed in the circumferential direction. This corresponds to a reinforcement ratio of about 0.24%, which is in close agreement with the requirement of a minimum of 0.25% horizontal reinforcement in walls to control cracking due to temperature and shrinkage as given in section 14.2.10 of ACI 318-77 (2).

In order to have prevented the formation of thermally induced cracks it would have been necessary to have used an internal piped cooling system to prevent a large temperature rise of the concrete, or perhaps the cone could have been cast in several smaller sections. The first alternative would probably have been an expensive proposition and the second alternative would probably have introduced serious construction problems.

In summary, the formation of cracks in the cones of the pier foundation was to be expected considering the massive nature of the structure and the construction sequence that was used.

QUESTION 1.2
Would the type of concrete mix used result in cracks?

RESPONSE
The characteristics and proportions of the ingredients in the concrete are some of the factors that affect the problem of controlling cracks due to restrained volume changes.

EXPLANATION
The interrelationships between a concrete mixture and the problem of controlling cracking are discussed in detail in Ref. (3); the key points are summarized in the following discussion. The two measures for controlling cracking of concrete are: 1) to produce a concrete having the best cracking resistance (tensile strain capacity); and 2) to control the factors which produce tensile strain in the concrete.

The tensile strain capacity of concrete is directly proportional to its tensile strength and inversely proportional to its modulus of elasticity. In addition, when the tensile strain is gradually applied, a high creep rate for the concrete is helpful in controlling cracking. For these reasons Ref. (3) makes the following statement concerning aggregate selection to achieve a concrete with a large strain capacity:

> "Where several sources of aggregate are available economically, preference should be given to that which yields best crack resistance; usually this will be a crushed material of low thermal expansion and low modulus of elasticity."

Thus the Florida limestone, which was used in the concrete and which is known to have a relatively low elastic modulus, may have been of some benefit in controlling the formation of cracks in the cone.

The other measure for controlling cracking is to minimize the induced tensile strains by limiting volume change and restraint. Volume change can be controlled by limiting the maximum temperature rise of the concrete at early ages. This can be achieved by:

- -using the lowest quantity of cement that is consistent with strength and durability requirements
- -using cement with low heat of hydration at early ages
- -using pozzolans as a replacement for some of the cement
- -using the lowest possible amount of mixing water
- -precooling the concrete during its production

Controlling restraint is a more difficult matter as a balance must be attained between minimizing restraint and keeping the construction simple.

The project specifications call for the following:

- -use of low-heat cement (ASTM Types II or IV)
- -use of 20 to 30% fly ash (a pozzolan) replacement for cement
- -use of water-reducing admixtures
- -use of ice for the mixing water

In conclusion, the project specifications included reasonable steps for controlling shrinkage induced cracking in the main piers.

QUESTION 1.3

FDOT acknowledges that some of the cracks may go all the way through the cone wall. What is the significance of cracks that go all the way through the cone wall?

RESPONSE

The cracks are not significant in terms of the structural capacity of the piers but they may be of significance in terms of long-term performance.

EXPLANATION

First, the philosophy of reinforced concrete design must be recognized. In designing a reinforced concrete member to safely carry a given external load, it is assumed that the portion of the member subjected to tensile stress is cracked and sufficient steel is provided to carry the necessary tensile loading. Hence, the presence of cracks in reinforced concrete members does not necessarily mean that structural capacity has been affected. To determine the significance, if any, of cracks in the cones it is necessary to consider the structural action of the cones and the relationship between crack orientation and load paths through the piers.

According to information provided by FDOT: "The cone acts primarily as a massive compression member... There are some local and bending effects also in the cone." This means that the cone walls carry an inclined axial force plus a bending moment. The structural drawings indicate a large number of #14 and #18 bars running in the vertical direction of the cones. These represent the reinforcement assisting the concrete in carrying the load from the superstructure down to the foundation. The circumferential bars in the cones are smaller (#8 bars) and the area of this reinforcement is 0.24% of the concrete area of a vertical section through the cone. As explained in the response to question 1.1, the amount of circumferential steel meets the minimum requirements for the control of shrinkage cracking. The adequacy of the circumferential steel to resist tensile stresses due to the external load on the piers depends on the magnitude of the circumferential stresses that may develop under the action of that load. It is our understanding that FDOT is in the process of performing a stress analysis of the piers. The results of that analysis should indicate whether the circumferential steel is adequate to carry any tensile stresses that may develop under service loads.

The cracks may have some significance in terms of the long-term performance of the piers. Specifically, the cracks may permit localized corrosion on some of the reinforcing steel. Although the bars are epoxy-coated, there are probably breaks in the coating which may have been produced during the placement and consolidation of the concrete. In addition, there could also be breaks that were produced during the placement of the steel which were not seen during the patching of the coating. If any of these defects in the coating happen to lie near the cracks, there is the possibility of localized pitting due to the presence of seawater. While such localized corrosion presents no short-term problems, it is not certain what may happen in the long-term. The basic problem is that epoxy-coated bars are relatively new products and we do not have sufficient historical records to determine the effects of small defects on long-term performance.

During the site visit to the piers it was observed that some of the cracks that were leaking water contained brown deposits along with lighter colored deposits. In one crack in the south pier the brown material could be seen coming out of the crack at the bottom of a core hole. The information that has been provided to us does not give a precise explanation of the nature of the brown colored deposits. It has not been shown conclusively that the brown color is not due to rust. An attempt should be made to secure samples of only the brown material so that it can be chemically analyzed. The results of the analysis should be compared with the analysis of deposits from other sources that are known to contain corrosion products. It then should be possible to determine whether the brown material is rust. It is also suggested that the efflorescence deposits should be cleaned so that the nature of any new deposits can be monitored.

In summary, the vertical cracks in the cones do not appear to have any significance in terms of the structural capacity of the piers, but they may have some significance in terms of long-term durability.

QUESTION 1.4

Are FDOT's plans to seal the inside cracks a reasonable solution, that is, will sealing the cracks make the water within the cone walls inert and thus not a threat to the rebars?

RESPONSE

FDOT should re-examine its current plan for sealing the cracks, giving careful consideration to the primary objective of any repair that is undertaken. It has not been adequately explained how sealing the cracks on one surface will make the water "inert".

EXPLANATION

As discussed in the response to question 1.3 the cracks pose an unknown factor with respect to future performance of the epoxy-coated steel. Thus it would be best to take a conservative approach and undertake remedial measures. The technique which is finally implemented should be consistent with the overall objective of the repair.

First, consideration should be given to the requirements for active corrosion of bare steel in concrete, which are as follows:

-a supply of oxygen,
-an electrically conductive path in the concrete, and
-depassivation of the steel.

If one or more of these items is eliminated, corrosion ceases.

Chloride ions present in seawater depassivate bare steel and may also increase the electrical conductivity of saturated concrete. Water-filled cracks in the concrete provide an access for chloride ions and dissolved oxygen to the steel. Because of diffusion, it is not necessary for the water to flow through the cracks in order for additional chloride ions and oxygen to migrate to the steel. Thus the objective of the repair method should be to keep additional chlorides or oxygen from reaching any bare spots on the epoxy-coated bars.

The repair method that is used should result in the filling of the existing cracks so that further penetration of chloride ions and oxygen is prevented. Because the repair is not intended to provide structural strength across the cracks, it is not necessary to use an epoxy adhesive type of material. The feassibility of using non-structural materials that have been developed exclusively for the purpose of sealing cracks should be investigated.

Therefore, it is suggested that careful consideration be given to the technique that will be used to repair the cracks in the cone walls. It is felt that the proposal to seal the cracks only on the interior surfaces of the cones is not the most appropriate approach, as this will not exclude the future migration of chloride ions and oxygen into the cracks.

QUESTION 1.5

When do or can shrinkage cracks become cracks of structural significance?

RESPONSE

Shrinkage cracks are of structural significance in a concrete structure when they reduce the load capacity or service life of the structure or if they prevent the structure from serving its intended function.

EXPLANATION

As was mentioned in the discussion of question 1.3, the design of a reinforced concrete structure assumes that the concrete will crack and reinforcing steel is provided to carry the tensile forces in the members. If a shrinkage crack develops in that portion of a member that will be subjected to tensile loading while in service, it is of no concern because the reinforcing steel will carry the tension. If a shrinkage crack develops in that portion of a member that will be subjected to compressive loading while in service, it is of no concern because the crack will close up under the compressive load. Thus shrinkage cracks in a properly designed reinforced concrete member are not structurally significant provided that cracking does not result in premature deterioration of the structure, such as through corrosion of the reinforcing steel.

There are many factors to consider when addressing the question of the significance of cracks on the durability of a concrete structure. One of the most important is the exposure conditions. Depending on the severity of the exposure conditions different crack widths can be tolerated. The following general guide for tolerable cracks widths has been adopted by ACI Committee 224 (3):

Exposure Condition	Tolerable Crack Width
Dry air or protective membrane	0.016 in
Humidity, moist air, soil	0.012 in
Deicing chemicals	0.007 in
Seawater and seawater spray; wetting and drying	0.006 in
Water retaining structures	0.004 in

Of the 18 crack width measurements reported by FDOT for the interior of the north and south cones, six exceeded 0.006 inches but none exceeded 0.010 inches. While the reported crack width measurements are not necessarily the maximum crack widths along each crack, they indicate that most of the existing cracks might be considered tolerable even for seawater exposure.

QUESTION 2.1
Does your agency have a position regarding the use of Florida limestone?

RESPONSE
The National Bureau of Standards has not carried out research on the use of Florida limestone in concrete, and thus has no position regarding the use of this aggregate.

QUESTION 2.2
Is FDOT's assertion that control of the mix and other materials offsets the limestone characteristics a reasonable one?

RESPONSE
Based on the reported concrete cylinder strengths for the piers, it is concluded that the control procedures adopted by FDOT have accommodated three of the four stated deficiencies of the limestone aggregates.

EXPLANATION
The concrete consultants' report identified four problems with using Florida aggregates:

1) variability of geologic formations in different parts of the state,
2) tendency for excessive degradation during handling, leading to the production of excessive fine material,
3) high and variable value of absorptivity, and
4) low strength and stiffness.

Because of the first three characteristics the consultants felt that by using Florida aggregate it would be difficult to control the variability of the properties of the finished product, that is, the concrete.

The following table was developed based on the cylinder strengths reported by FDOT for the various placements in the two piers:

Element	Required Strength (psi)	Average Measured Strength (psi)	Standard Deviation (psi)	Coefficient of Variation (%)
N. Seal	3000	5670	357	6.3
S. Seal	3000	4040	216	5.3
N. Shafts	3400	4990	524	10.5
S. Shafts	3400	4720	491	10.4
N. Bot. Ring	4200	5560	382	6.9
S. Bot. Ring	4200	5135	258	5.0
N. Cone	4200	5120	322	6.3
S. Cone	4200	5425	366	6.7
N. Top Slab	4200	5680	403	7.1
S. Top Slab	4200	5450	324	5.9

According to ACI Standard 214-77 (4), for general construction testing, a standard deviation less than 400 psi corresponds to excellent control and a standard deviation between 400 and 500 psi represents very good control. Thus it is clear that the procedures implemented by FDOT have produced concrete with consistent strength properties.

Concerning the question of low aggregate stiffness, it is not possible to compensate for this characteristic. However it was found that the elastic modulus of concrete made with the Brooksville aggregate was reported to be about 4,000 ksi. This measured value agrees with that predicted by the ACI formula which considers the unit weight and compressive strength of the concrete. Thus it is concluded that the concrete made with the Brooksville aggregate has the magnitude of elastic modulus expected for its strength and density.

QUESTION 2.3
If the recommended Georgia and Alabama aggregate had been used, would the quality of the concrete be measurably better? How?

RESPONSE
It is uncertain whether the use of the Georgia or Alabama aggregates would have, in itself, resulted in concrete of significantly higher quality than that obtained with the Florida limestone.

EXPLANATION
First, the meaning of "quality concrete" needs to be defined. Quality concrete is concrete which has the required properties in both its fresh and hardened state, which is economical, and which has an acceptable degree of variability. From this definition, it is seen that what is, and what is not, "quality concrete" depends on the specifics of each project. The production of quality concrete requires both the use of acceptable materials and the use of sufficient controls to assure that the materials are correctly handled and proportioned.

Judging from the average values and standard deviations of the reported cylinder strengths, it would have to be concluded that the concrete used in the piers is of good quality. It is uncertain that a better quality concrete would have resulted if the Georgia or Alabama aggregates had been used. The harder aggregates might have resulted in slightly higher compressive strengths for the same amount of cement (cement factor). This could have resulted in using a lower cement factor which could have reduced the temperature rise in the concrete placements. On the other hand, had the harder aggregates been used, it is possible that a different control program would have been used which could have produced concrete of higher variability than was attained with the limestone. With the harder aggregates, the resulting concrete would have had a higher elastic modulus and reduced creep potential. While this may have been of some benefit for the design of the superstructure, it would not have been a benefit in controlling cracking in the cones due to induced tensile strains. Thus it cannot be concluded with certainty that use of the Alabama and Georgia aggregates would have produced a better quality concrete.

QUESTION 2.4

Can the actual aggregate properties of creep and shrinkage be incorporated into the design as MIT suggests and are the creep and shrinkage factors for concrete made with Florida limestone known?

RESPONSE

Yes, the actual creep and shrinkage properties of the concrete can be incorporated into the design, and the necessary data are available.

EXPLANATION

For prestressed concrete structures, a key element in their design is the consideration of the time-dependent deformations due to creep and shrinkage. These have to be considered, for example, to predict the deformation of the structure and to determine the required level of initial prestressing forces. A textbook on the subject of prestressed segmental bridges states that (5):

> "Reliable data on the potential of the mix in terms of strength gain, creep, and shrinkage performance should be developed to serve as the basis for improved design parameters."

Thus for design purposes, good engineering practice requires knowing the properties of the concrete to be used in the structure.

Creep and shrinkage data of concrete made with Florida aggregates were developed for FDOT by the Waterways Experiment Station of the Corps of Engineers. In addition, representatives of FDOT have stated that Construction Technology Laboratories of the Portland Cement Association and the University of Florida are also generating data on concrete with the aggregates used in the bridge construction. It is also our understanding that FDOT is requiring the designer to update the original design by taking into account the available creep and shrinkage data.

In summary, the creep and shrinkage properties of the concrete being used to build the the bridge can and should be incorporated into the final design of the superstructure.

QUESTION 2.5
What is the relationship, if any, between the decision to use Florida limestone and the cracks in the concrete?

RESPONSE
There is no significant relationship between the use of the Florida limestone and the formation of the cracks in the cone walls.

EXPLANATION
The explanation for the causes of the cracks in the cone walls and the effect of the concrete mixture on the formation of cracks was given in the discussions of questions 1.1 and 1.2. To repeat, cracks occur when the induced tensile strains resulting from restrained volume change exceed the tensile strain capacity of the concrete. In the case of the cones, the volume change was brought about by the temperature rise of the mass concrete and the restraint was due to the presence of the bottom ring. Had Florida aggregates not been used, there probably would not have been any significant changes in these two factors and, all other conditions being the same, the cracks would still have developed.

QUESTION 3.1
Should the concrete mix formula used in the main pier foundations result in quality and durable concrete?

RESPONSE
Based on the available information, the concrete used in the piers is considered as being of good quality and it should perform well.

EXPLANATION
As mentioned in the discussion of question 2.3, quality concrete results when acceptable materials are used and when the materials are handled and proportioned in a proper manner. In this particular situation, several specific actions were taken by FDOT to ensure a quality concrete:

1) use of low-heat cement with limitations on the contents of alkalies and tricalcium aluminate,
2) use of entrained air and a water reducing admixture,
3) use of ice for mixing water to lower the initial concrete temperature,
4) use of fly ash to replace some of the cement,
5) use of a low water-cement ratio,
6) limiting of the approved sources for coarse aggregate, and
7) requiring water sprinkling of the coarse aggregates to control absorption.

The results of tests by Construction Technology Laboratories indicated that the limestone aggregate could be considered innocuous with respect to reactivity with alkalies in cement. Thus it is unlikely that there would be any durability problems because of the alkali-aggregate reaction.

The results of the cylinder strength tests, as discussed in the response to question 2.2, suggest that very good to excellent quality control was used to produce the concrete. In summary, there are no indications that the concrete is of poor quality or that it will have poor durability.

QUESTION 3.2

Would the five "above normal" steps taken by FDOT enhance the concrete?

RESPONSE

The five steps, plus others that were implemented, are positive approaches for enhancing the expected performance of not only the concrete but also the structure.

EXPLANATION

It is our understanding that the five steps in question are as follows:

1) use of cement having low heat of hydration, low alkali content and resistance to sulfate attack,
2) use of fly ash to replace some of the cement,
3) use of ice,
4) use of epoxy coated reinforcing steel, and
5) use of a penetrant sealer on exposed concrete surfaces.

As discussed in the response to question 3.1, the first three items are helpful in producing durable concrete. The use of epoxy coated bars and the sealer are helpful in minimizing the possibility of rebar corrosion. Another deterrent to corrosion is the 4 inches of concrete cover used on the steel in the piers.

In summary, it appears that FDOT has taken reasonable steps towards producing a durable concrete and a durable structure.

QUESTION 3.3

Is the 2-hour pour time a general rule-of-thumb time and is a 28-hour set time feasible with the mix used?

RESPONSE

To our knowledge, there is no rule-of-thumb time limit on the placing of a drilled shaft. The information provided us indicates that the room temperature initial setting time of a concrete mixture similar to that used for the shafts would be expected to be greater than 6 hours.

EXPLANATION

The probable reason for putting a time limit on the placement of the concrete in the drilled shafts is so that there would be no possibility of developing cold joints within the shafts. A cold joint is the discontinuity that develops if the next layer of concrete is placed on a previous layer that has already undergone setting, thereby precluding a union of the concrete in adjacent layers. Hence the important parameter is not the elapsed time for the placement of the entire shaft but rather it is the elapsed time between the placement of adjacent layers.

The setting time of a concrete mixture is dependent on many factors. Some of the more important ones include the concrete temperature, the initial consistency, and whether admixtures are used. At room temperature the initial setting time of concrete would be expected to be within 4 to 6 hours. However, the concrete used in the shafts would be expected to have a setting time longer than 4 to 6 hours for the following reasons:

- -an admixture was used,
- -the initial concrete temperature was lower than room temperature, and
- -the slump was high.

Based on the information provided us, it is unlikely that the setting time of the concrete in the shafts was as high as 28 hours. The long setting times reported by FDOT were for concrete mixtures with a set retarding admixture. However, the admixture used in the shafts, referred to as LL-819, does not appear to be a set retarding admixture. Manufacturer's literature states that "concrete with admixture LL-819 sets at a rate comparable to plain concrete..." It is further stated that "Admixture LL-819 is recommended for use in all concrete where normal setting characteristics are required or desired." Test data provided on the performance of LL-819 indicate that at a dosage rate of 2-1/2 fl oz/100 lb cement and for concrete having a 2-3/4 inch slump, the initial setting time is 6 h 10 m compared with 5 h for a concrete without the admixture. The concrete in the shafts contained the admixture at a dosage of 5 oz/100 lb of cement, which would be expected to cause additional retardation of setting. For this reason and because the concrete had a high slump and an initial temperature below the standard used to determine setting time, it is concluded that the concrete in the shafts would have had an initial setting time in excess of 6 hours. However, it is unlikely that the initial setting time would have been as high as 28 hours.

QUESTION 3.4

What is the significance of the 18 pourings exceeding the 2-hour specification (no pouring exceeded 4 hours) in light of the 28-hour set time?

RESPONSE

There is no significance to the fact that the elapsed times for the placement of 18 of the shafts exceeded the 2-hour specification limit.

EXPLANATION

As discussed in the response to question 3.3, the real concern is the elapsed time between the placement of adjacent layers in a shaft, rather than the time to place the entire shaft. According to the data provided us, none of the 18 shaft placements in question exceeded 4 hours, and 15 of them were placed within 2-1/2 hours.

Because the concrete contained an admixture which had some retarding effect and because the concrete was placed at a lower than normal temperature, it is unlikely that initial setting had begun in the shafts even after 4 hours. Thus there would be no adverse effects to those shafts that surpassed the 2-hour limit.

4.0 CONCLUSIONS

Based on our study of the information provided us by GAO and the additional information obtained from FDOT, the following conclusions are drawn with respect to the three concerns addressed by NBS:

1. Significance of cracking in piers

The cracks pose no immediate concern with respect to the structural capacity of the main piers. The cracks are probably the result of restraint to the thermal shrinkage due to the temperature rise and subsequent cooling that occurred in the massive concrete placements. The Florida Department of Transportation took reasonable steps in preparing the job specifications to minimize the temperature rise of the concrete placements. It would have required a much more complicated construction procedure to have totally eliminated the formation of thermal shrinkage cracks. However, since the cracks can permit the intrusion of seawater, it is prudent that they be sealed. The intrusion of seawater, which contains chloride ions and dissolved oxygen, can lead to localized corrosion of the reinforcement at the location of any defects in the epoxy coating.

2. Materials used in the concrete

The reported cylinder strength results for Contract 1 show that the concrete produced with the Florida limestone aggregates had low variability and average strengths well above the design strengths. Thus there is no question that the concrete is adequate from a strength point of view. Since the aggregates are innocuous with respect to reactivity with alkalies in the cement and since fly ash and a low water cement ratio were used, the concrete is expected to be durable. Concerning the creep and shrinkage behavior of the concrete, FDOT is making efforts to obtain additional data that can be incorporated in the final designs of the superstructure. In addition, there are plans to instrument the completed structure in order to monitor its time-dependent behavior.

3. Procedures used in placing concrete in the drilled shafts

The concern that some of the shafts may be weakened because the concrete placement took longer than the 2-hour specification limit is unfounded. Considering that the important elapsed time is that which occurs between the placement of adjacent layers and not that which occurs between the start and completion of the entire shaft placement, and considering that the initial setting time of the concrete was probably in excess of 6 hours, it is highly unlikely that cold joints developed.

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11. ABSTRACT (A 200-word or less factual summary of most significant information. If document includes a significant bibliography or literature survey, mention it here)

The General Accounting Office (GAO) requested the assistance of the National Bureau of Standards in the investigation of the construction of the new Sunshine Skyway Bridge in Florida. Specifically, GAO desired answers to questions related to the following: 1) the formation of cracks in the main piers of the bridge span; 2) the materials used in the concrete mixtures; and 3) the procedures used in the placement of concrete in the drilled shaft foundations. The objective of the GAO inquiry is to determine the reasonableness and validity of the positions taken by the Florida Department of Transportation on each of the concerns expressed by a number of individuals in connection with the bridge construction. This report provides answers to the questions and provides explanations for each answer.

It is concluded that the cracks in the piers will not affect the structural capacity of the piers. However, the cracks should be sealed as a precaution against the intrusion of additional seawater, which may cause localized corrosion of the reinforcement at any existing breaks in the epoxy coating on the reinforcing steel. The coarse aggregates that were used appear to have resulted in a good quality concrete which should be durable. Finally, there is no basis for concern with regard to the integrity of the drilled shafts.

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