57 YEARS OF COASTAL ENGINEERING PRACTICE AT A PROBLEM INLET: INDIAN RIVER INLET, DELAWARE.

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INTRODUCTION

The Army Corps of Engineers constructed twin, parallel jetties at Indian River Inlet in the period 1938-39. This construction represented the culmination of almost 50 years of intermittent interest expressed by citizens of Delaware to obtain a safe, stable passage from Indian River and Rehoboth Bays to the Atlantic Ocean. However, even before the seaward ends of the jetties were complete, the <u>first</u> of several coastal engineering problems arose as the channel banks between and west of the jetties began to erode and threatened to flank the landward ends of the structures. The <u>second</u> significant problem at Indian River Inlet became evident by the early 1950's as the ocean beach north of the inlet progressively eroded and threatened to breach the State highway along Delaware coast. The <u>third</u> but least visible problem to arise at the inlet became evident in the late 1970's as the channel between the jetties experienced accelerated, generalized scour which threatens the structural integrity of the jetties.

Engineering solutions have been applied to the interior shoreline and north ocean beach shoreline erosion problems, with the result that the erosion has been controlled with revetments and a sand bypassing system, respectively. However, inlet scour continues and presently poses what may prove to be the most difficult and costly of the coastal engineering challenges presented in the 100 plus years since locals first petitioned the Government for a jettied inlet.

This paper presents a review of Indian River Inlet's behavior in its natural state prior to jetty construction and summarizes the level of understanding of coastal processes and inlet-bay tidal hydraulics as seen from the perspective of the coastal engineers/designers working to improve the inlet in 1935. The paper also reviews the types of analyses applied in developing remedial plans for the interior and ocean shoreline problems, and summarizes the performance of the erosion control features which have been constructed.

It will be shown that qualitatively the coastal engineers working on this project in 1935 recognized the same parameters which today's coastal engineers would identify as integral components of the design process for jetties at a tidal inlet: longshore transport and the potential for downdrift beach erosion; inlet-bay tidal hydraulics, inlet current velocities, and bay

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tide range and salinity; and cost to stabilize the inlet and the benefits to be derived therefrom. Yet it is also clear that the actual behavior of the inlet following jetty construction and initial channel dredging has been far more problematic than its designers anticipated. Would the generation of coastal engineers practicing almost six decades later do a better job of designing and constructing a stable passage from the ocean to the inland bays and yet avoid the interior and exterior shoreline erosion and channel scour problems which developed during and after construction?

DESCRIPTION OF THE STUDY AREA

The Atlantic Ocean coast of Delaware consists of approximately 25 miles of sandy shoreline extending from Fenwick Island at the Delaware-Maryland state line north to Cape Henlopen at the entrance to Delaware Bay. (See Figure 1.) The shoreline approximates a straight north-south line, and its continuity is interrupted only by Indian River Inlet, located midway along the coast. The shoreline is exposed to the essentially unlimited fetch of the open Atlantic to the east, such that the physical processes of the shoreline are dominated by ocean waves, although it will be shown that there are important localized effects related to the tidal circulation of Indian River Bay and also Delaware Bay. The shoreline is also subject to semi-diurnal tides with an ocean mean range at Indian River Inlet of about 4 ft, and a spring tide range of almost 5 ft. The existence of tidal current effects has been recognized qualitatively for decades, but no investigation to date has resolved the relative magnitude of tidal current effects on longshore transport as compared to wave effects.

Indian River Inlet provides the only direct connection from Delaware's inland bays to the Atlantic Ocean. Indian River Bay and Rehoboth Bay are nearly equal in surface area with a combined total of approximately 29 square miles. The total area tributary to Indian River Inlet, including the bays' surface, is about 250 square miles. The mean depth of both bays at low water is about 5 ft. Freshwater input to the bay system yields salinities which typically range from about 15 to 25 ppt away from the immediate influence of the inlet. Indian River and Rehoboth Bays presently experience mean tide ranges which average 2.1 and 1.0 ft, respectively, based on tide gage observations from 1984 to the present. The hydraulic characteristics of Indian River Inlet are thus strongly dominated by tidal forcing, with secondary effects due to local winds on the bays and over the adjacent shelf. Freshwater is considered to have negligible influence on flow at the inlet.

At Indian River Inlet the predominant longshore transport direction is northward, as evidenced by accretion of the beach south of the south jetty and erosion north of the north jetty since 1938-39. Northward longshore transport is predominant in the shoreline reach from Indian River Inlet north to Cape Henlopen. However, at Ocean City (Maryland) Inlet, located 20 miles to the south, predominant transport is clearly to the south as shown by accretion on the north (Ocean City) side of the inlet and significant erosion south of the inlet on Assateague Island. In the reach between Indian River and Ocean City Inlets there is thus a reversal in the direction of predominant longshore transport.

HISTORY PRIOR TO JETTY CONSTRUCTION.

Reliable coast surveys dating to 1843 and other historic accounts show that the inlet location has varied over a 2 mile-long zone centered on the present location. Between 1843 and 1928, the natural inlet opening was unstable in location and tended to close occasionally, as littoral processes delivered more sediment to the inlet throat than tidal currents were able to scour away. The inlet regime was thus dominated by littoral processes. After closures, the inlet reopened naturally only after the bay water rose high enough to breach the narrow barrier from the west, or when the ocean, due to a combination of waves and tide, breached the barrier from the east.

In the period between 1928 and 1937, state and local interests made several attempts to open and maintain an inlet across the barrier by dredging alone. However, each of these

attempts was at best only temporarily successful, as there were no provisions for structures to prevent the influx of littoral sediments to the channel.

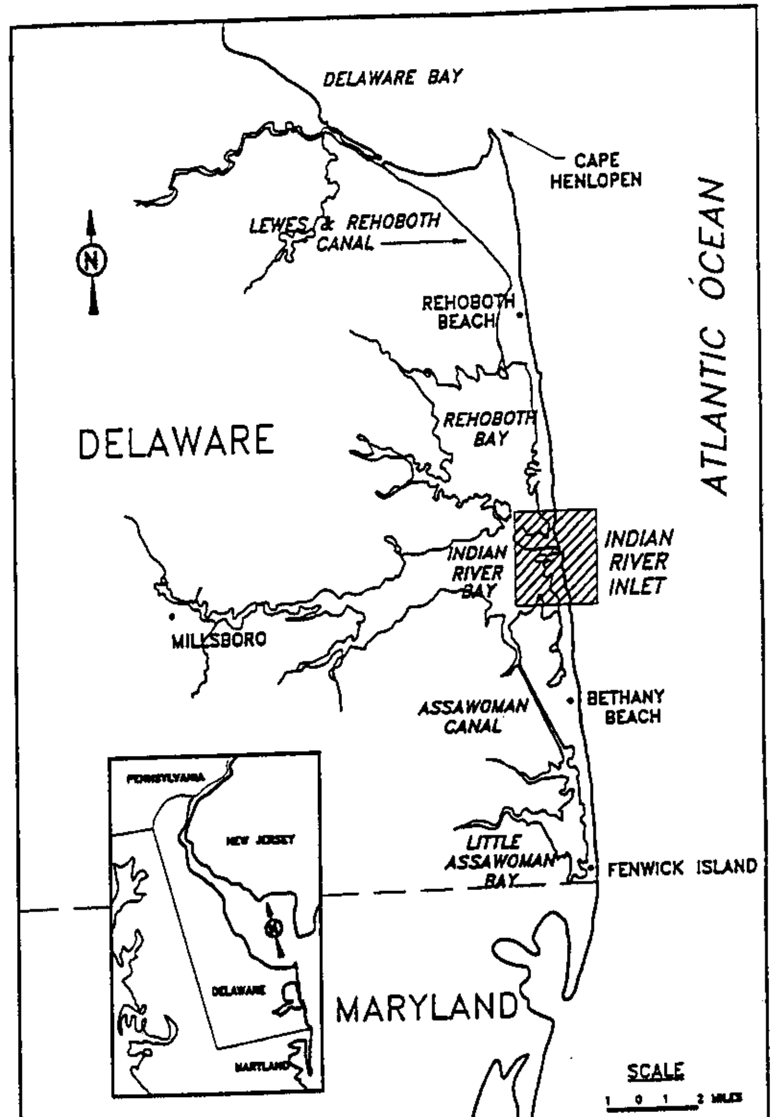


FIGURE 1. VICINITY MAP.

STUDIES LEADING TO JETTY CONSTRUCTION, 1935 - 1937.

With the failure to keep Indian River Inlet open by dredging alone, local interests prevailed on Congress to authorize a study in 1935 to determine if a stabilized inlet was feasible and justifiable. Improvements were desired for several purposes, including better navigability for both small commercial as well as recreational vessels, and restoration of tidal circulation and salinity to encourage fish and shellfish species which could not tolerate the water quality effects associated with frequent inlet closures.

The Philadelphia District conducted inlet design studies in 1935 and 1936, and review comments and suggested modifications were provided by the Shore Protection Board in 1936. These studies were conducted by some of the pioneers of coastal engineering within the Corps of Engineers, including Mr. Clarence Wicker of the Philadelphia District, who later worked to establish the Committee on Tidal Hydraulics, and Colonel Earl I. Brown, senior member of the Shore Protection Board and author (in 1928) of "Inlets on Sandy Coasts".

The technical analyses performed in support of the effort to stabilize Indian River Inlet concentrated on two principal topic areas: the potential effects of jetty structures on the adjacent beaches, and the tidal hydraulic characteristics of a jettied channel connecting the large, shallow inland bays to the Atlantic Ocean.

Longshore Transport. The evaluation of longshore transport in the vicinity of Indian River Inlet was based on observation of currents in the nearshore zone and on examination of shoreline response to groins in communities north and south of the proposed inlet location. The current observations were obtained with drogues and current meters over a limited number of tidal cycles under conditions of low wind and wave effects. Results of these observations showed that the nearshore current direction, out to a distance of about 2,000 ft from the shoreline, varied according to the rise and fall of the ocean tide. Rising tides were accompanied by northward currents, and falling tides were accompanied by southerly currents, presumably due to the effects of the flood and ebb of tidal flow at the entrance to Delaware Bay to the north.

The groin fields located at Rehoboth Beach north of the inlet and at Bethany Beach south of the inlet were examined as part of the inlet design studies. These observations showed a slight tendency for accretion on the south side of the groins and erosion on the north side both at Rehoboth and Bethany. It was thus concluded from the groin and current observations that longshore transport at the proposed location for the jetties was predominantly southward in winter and northward in summer, with only a small net northward transport on an average annual basis. The report by the Shore Protection Board stated that construction of jetties would "reduce the net volume of drift now passing the inlet from south to north and will, consequently, slightly reduce the supply now nourishing the northerly beaches". Given the lack of development for several miles north of the inlet, the reduction in sediment supply was not viewed as a serious potential problem.

Inlet Tidal Hydraulics. The hydraulic design principles applied by the District in 1935-36 were based on procedures presented by Col. Brown in "Inlets on Sandy Coasts". In this analysis, it was assumed that inlet flow is induced by the ocean tide variations and could be treated as open channel, uniform cross-section flow driven by the surface slope from the seaward end of the jetties to the interior end of the inlet channel where it joins open water of the bay. The two interior bays were assumed to have a uniform tide range for any given inlet channel geometry and ocean tide range.

The Chezy equation,

$$v = C\sqrt{r \times s}$$

was used to determine the maximum cross section-average velocity assumed to occur when the water surface slope from ocean to bay (or bay to ocean) was at its maximum. In this equation, "v" is velocity, "r" the hydraulic radius, and "s" the surface slope. The value of "C" was determined from the Kutter-Gangiullet formula (derived in 1869) which includes the terms "r", "s", "n" (a roughness coefficient, numerically equivalent to Manning's "n"), as well as several empirically determined constants. The computed maximum velocity for any given set of channel dimensions and ocean tide range was then applied to another of Col. Brown's formulae,

$$Q = 17{,}050 \times a \times v_{max}$$

to compute the tidal prism "Q" in cubic feet, with "a" being the inlet cross-section at mid-tide and "17,050" having units of seconds. The procedures used to compute "v max" and "Q" were as follows. For each alternative channel cross-section, a trial value of bay tide range was assumed, with the maximum water surface slope (s max) calculated as one-half of the square root of the difference of the squares of the ocean tide range and the trial bay tide range, divided by L, which is the design channel length of 5850 ft. The ocean tide range used in all computations was the mean range for the study area, 4.0 ft. The procedure used to calculate the "s max" value is physically representative of the condition which would occur with the ocean tide at its maximum (or minimum) value at a time when the bay tide was approximately at its mid-range value.

The channel friction coefficient "n" was determined to be 0.045 from prototype measurements of the pre-jettied channel in 1935, with an assumed improved condition "n" of 0.035. The Chezy equation was then solved for "v max". The value of "v max" was then substituted into the equation for "Q" to obtain the tidal prism. This tidal prism divided by the total bay surface area yielded a bay tide range. If the resultant bay tide range differed appreciably from the trial value, a second value was assumed and the calculations with the Chezy and tidal prism equations repeated until satisfactory agreement was obtained between the trial and computed bay ranges. A range of channel dimensions, from a minimum of 250 x 8 (ft) to a maximum of 500 X 10 (ft), was evaluated using these procedures. The resultant bay tide ranges were computed to be in the range of 0.21 to 0.56 ft, with v max values between 3.5 and 4.2 ft/sec. Although it was understood that the considered channel improvements would lead to a greater tidal exchange between ocean and bays, there was no quantitative evaluation of the potential salinity increases in Indian River and Rehoboth Bays.

With the advantage of hindsight, the design v max values in the 3.5 - 4.2 ft/sec range would appear to provide a clue to the subsequent generalized scour behavior at Indian River Inlet. It was recognized in the 1935-36 channel design work that these velocities were capable of transporting sediment sizes up to and including coarse sand, despite the fact that littoral sediment sampling and grain size determinations showed the native sediment size in the study area to be medium sand. It should also be noted that these velocities were computed to occur under conditions of mean ocean tide range, with no apparent consideration given to potential velocities under spring tide or storm surge conditions. Scour was simply not considered to be a problem; the principal emphasis was the desire to minimize or eliminate maintenance dredging requirements. In defense of the inlet's designers, it should also be recognized that at that time there were no known examples of jettied inlets at which serious, generalized scour had occurred. There are few examples of this type of behavior even today. Suffice it to say that there has never been a maintenance dredging burden in the channel between the jetties.

It is interesting to note that the original (12/35) Philadelphia District design proposals for the jetty-to-jetty spacing called for a 600 ft width. However, this width was reduced to 500 ft by the recommendations of the Shore Protection Board in August 1936, in the interest of reducing potential shoaling and maintenance dredging within the inlet. It presently remains a matter of speculation whether the 600 ft width would have led to a significantly different course of inlet channel evolution than has actually occurred. Given the severity of the scour problem (to be discussed more thoroughly later in this paper) it is would appear unlikely that a 100 ft increase in intra-jetty spacing would have altered the inlet hydraulics sufficiently to offset the existing scour trend.

JETTY CONSTRUCTION

Construction of the twin, parallel jetties began in 1938. The final design adopted for construction included a center-to-center spacing of 500 ft and total length for each jetty of approximately 1500 ft, extending to the then 14 ft ocean MLW contour. The outer 600 ft of each structure was built entirely of stone, with the inner 900 ft constructed as a bulkhead with steel sheet piling. (See Figure 2 for a recent (4/90) aerial photograph of the inlet. LettMwed locations shown on the figure are referenced in the text.) Channel dredging was performed in 1938 while the jetties were still under construction, and was immediately followed by scour along the bulkheaded sections. Stone toe protection was added to the channel side of the bulkheads in 1939-40, with a tie-rod and anchor system added later in 1940.

As originally designed, the jetties consisted only of the 1500 ft parallel structures (A to B, Figure 2), with no landward extensions envisioned. However, tidal currents at the landward end of the jettied section led to lateral scour and erosion of the unstabilized channel banks, threatening to flank the approaches to the highway bridge and the support towers for the electrical transmission lines.

In order to arrest this erosion, a sequence of landward extensions to the original jetty structures was initiated in 1941. The first increment consisted of bulkheads constructed with 45 degree flares from the alignment of the jetties, widening the shoreline from the 500 ft jetty spacing to an 800 ft spacing back to a point about 900 ft west of the original landward terminus of the jetties (B to C, Figure 2). Even before completion, these bulkheads were being flanked at their western ends, and stone dikes about 250 ft long were added with another 45 degree widening flare (C to D, Figure 2). These dikes were soon flanked, and this was followed by bulkhead failure on the south side of the inlet in 1946. Extensive remedial work was initiated in 1947, with more stone toe protection added to the channel side of both the south and north bulkheaded sections west of the original jetty construction, plus additional tie backs and anchors for the bulkheads. By the time this work was completed, the original jetty lengths of 1500 ft had each been extended landward along an additional 1200 lineal ft of inlet interior shoreline.

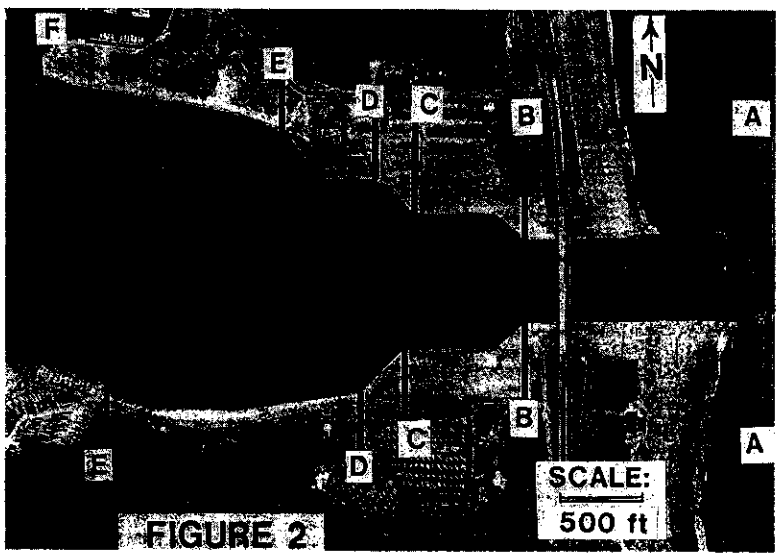


FIGURE 2

The bulkheaded sections west of the original jetty structures performed satisfactorily to stabilize the shoreline through 1962, when additional stone was added along both north and south bulkheads to offset corrosion damage to the steel bulkheads. A major rehabilitation of the bulkheads was performed in 1973, when the bulkheads with toe protection were essentially replaced by full stone revetments. This resulted in the condition that the entire inlet throat, including the original 1500 lineal ft of jetty and the approximately 1200 additional lineal ft of interior shoreline, was armored with stone from the structure crest to the toe, typically at depths of from -5 to -20 ft MLW.

INTERIOR SHORELINE EROSION

During the period from 1941 to 1973, when the interior shorelines immediately west of the original jetty structures were being stabilized, the adjacent unstabilized portions of the north and south shorelines continued to erode as the channel widened further. During this same period, the previously undeveloped land adjacent to the inlet was progressively undergoing transformation, with the construction of a campground, trailer park, and a private marina on the south shore, and a campground, U.S. Coast Guard Station, and a State-operated marina on the north shore. Erosion rates, as measured by shoreline retreat relative to the channel centerline, were on the order of 15 ft/yr along 4,000 lineal ft of the north shoreline (1938-79), and somewhat lower, about 10 ft/yr, along the 3,000 lineal ft of the south shoreline (1938-68). Erosion of the south shoreline was significantly reduced beginning about 1968 by the dumping of construction rubble by the State of Delaware.

In order to address erosion problems of the interior shorelines, the Philadelphia District conducted studies in 1984 and 1985 under the authority of the Section 103 program (small project shoreline erosion control). The cause of the erosion was determined to be the combined effects of bay-generated waves and tidal current velocities through the inlet. A range of engineering improvements was evaluated in detail for the north and south interior shorelines, including stone, gabion, and grout-filled nylon bag revetments, steel bulkhead, and beachfill. The critical design parameter for the shoreline stabilization was a design wave height of 3.5 ft, based on fetch limited breaking waves generated over Indian River Bay to the west. Based on an economic evaluation, the stone revetment was selected as the recommended plan.

Construction of the north and south interior shoreline protection projects was accomplished in 1988 and 1989. The principal features of this work included approximately 1600 lineal ft of revetment on the south shore and approximately 1900 lineal ft of revetment on the north shore (D to E, Figure 2). The revetment cross sections were identical on the north and south shores, with a graded rip-rap cover layer 3 ft thick comprised of 90 - 1500 pound stone. Suitable underlayer and bedding layers were also provided. The structures have a crest elevation of +5 ft NGVD (approx. +6 MLW) and a toe elevation of -6.5 ft NGVD (-5.5 MLW). These structures have performed satisfactorily to date.

NORTH OCEAN BEACH EROSION

As reported earlier in this paper, it was perceived during project design that there was a small but unquantified net northward longshore transport at Indian River Inlet, and that any interruption of sand transport caused by the jetties would be minor and inconsequential due to the lack of development north of the inlet. The State highway was the only improvement for at least two miles north of the jetties, and it was located parallel to and about 700 ft landward of the low water line. However, downdrift shoreline adjustment began soon after completion of the jetties. By 1954, the 3,000 lineal ft of beach north of the inlet had eroded such that it was characterized by a dune scarp less than 200 ft from the roadway in places. The first of eight major beachfills to the present was placed in 1957, when the State dredged about 0.5 million cubic yards from a backbay site north of the inlet.

For the next 25 years, erosion of the north ocean beach became the dominant coastal engineering problem associated with Indian River Inlet. In this period, backbay borrow sources north of the inlet proper were utilized three times, in 1957, 1963, and 1982, with a total of about 1.3 million cubic yards placed on the north ocean beach. Then in 1972, the first major beachfill borrow operation utilizing the inlet flood shoal was conducted. Between the first operation in 1972 and the most recent in 1990, a total of about 2.3 million cubic yards of flood shoal sediment has been dredged and pumped onto the north ocean beach, typically along the 3,000 lineal ft of beach immediately north of the north jetty, up to a maximum of 5,000 ft north of the inlet. This dredging has been performed in a zone extending about 4,000 ft west of the highway bridge over the inlet, through a combination of both State and Federally funded projects.

Ironically, in 1965 as the north beach erosion was revealing itself as a persistent and costly phenomenon, the construction of the new State highway bridge over the inlet increased the immediacy of the problem. The new bridge piers were constructed about 250 ft seaward of the old bridge in order to avoid the considerably larger inlet span required to the west of the old bridge, where the earlier interior shoreline widening had occurred. The roadway north of the inlet was thus displaced as much as 300 ft seaward of its original alignment, leaving an even smaller buffer of beach between the now convex-seaward roadway and the shoreline.

The Philadelphia District initiated studies in 1983 to evaluate the erosion problem of the north ocean beach, and to determine if there was a more efficient alternative than the large, infrequent beachfills employed since 1957. The principal objective of the coastal engineering analyses of this study was to develop an improved understanding of sediment transport for the Indian River Inlet vicinity, including the mechanisms and rates of sediment loss on the north ocean beach, as well as the roles played by the flood and ebb shoals as sediment sinks. The goal of this work was to determine the volume rate at which sediment should be provided to the north ocean beach to provide a stable buffer between the State highway and the shoreline.

A number of approaches were applied in the attempt to quantify longshore transport rates at Indian River Inlet. One method involved the application of Phase III WIS statistics to the SPM longshore transport rate equation. The initial attempt utilized WIS data tapes for Atlantic Coast Station 66, which is the station closest to Indian River Inlet. However, this analysis resulted in a predicted net southerly transport of about 60,000 cy/yr, clearly opposite to the net transport direction demonstrated by the ocean shorelines adjacent to the jetties. A second attempt utilized the data from WIS Station 65, located north of the inlet, and resulted in a more physically reasonable value of about 160,000 cy/yr to the north, with a standard deviation of 90,000 cy/yr. It is believed that the results from the Station 66 simulation are inappropriate for the inlet vicinity, and probably reflect the effects of transport in the previously discussed sediment transport nodal zone located to the south between Indian River and Ocean City Inlets.

A second approach used to evaluate longshore transport at Indian River Inlet was based on analysis of the observed changes in hydrography in the general vicinity of the inlet over the period 1939 to 1980. The most significant morphological change, other than the ocean shoreline adjustments, was the development and growth of the ebb shoal. Hydrographic surveys performed prior to construction of the jetties typically show depth contours offshore of the inlet which are straight and parallel to the then-existing shoreline. The absence of an ebb shoal for the inlet in its pre-jetty condition reflects the dominance of wave processes over inlet tidal hydraulic processes.

However, a study by Collins (1982) demonstrated that following completion of the jetties, an ebb shoal developed and accumulated sediment at the rate of about 120,000 cy/yr. The ebb shoal is the most significant sediment sink in the inlet vicinity. The sediment deficit of the north ocean beach is believed to be directly related to the growth of the ebb shoal. The inlet in its present configuration forms a near total barrier to natural sand bypassing across the inlet mouth. The interruption in natural bypassing is caused by the combined effects of the jetties, especially the south jetty which diverts the larger northward transport offshore, and the interaction of the inlet and ocean tidal currents which lead to deposition on the ebb shoal.

The third approach was based on analysis of the behavior of the north ocean beach during the period following the first beachfill in 1957. In this period, the annualized rate of beach nourishment was about 105,000 cy/yr. Although the north beach experienced large variations in width in this period due to an average interval between fills of about 5 years, with large beachfill-related advances followed by persistent erosion losses, the average rate of fill placement was sufficient to prevent any net loss compared to the starting conditions of 1957.

The results of these analyses were synthesized and led to the adoption of the value of 110,000 cy/yr as the net northerly transport at Indian River Inlet, and also formed the basis for the projected requirement of 110,000 cy/yr as the north ocean beach sediment deficit. A number of alternatives were considered to address this problem, including bridge and roadway relocation, and construction of a hard protective structure along the north ocean beach. The study also evaluated several periodic nourishment alternatives, including trucking sand from the south jetty fillet or other off-site borrow area, conventional dredging of the south jetty fillet or nearby shoals, and sand bypassing from the fillet area with a jet pump. Based on an economic analysis of these alternatives, the jet pump sand bypass system was selected on the basis of greatest net benefits.

SAND BYPASSING SYSTEM

The sand bypassing system was designed around the requirement to transport an average annual quantity of about 110,000 cy/yr from the south jetty fillet to the eroding north ocean beach. System design also required that constraints posed by assumed available pumping time on a daily, weekly, and seasonal basis be considered in order to arrive at an optimum pumping rate capacity. A study performed by Weggel and Douglass (1986) utilized synthetically generated daily longshore transport rates at Indian River Inlet, together with a range of assumed south beach storage capacities and daily pumping rates, to determine the optimum pump capacity. It was determined that a jet pump capacity of 200 cy/hr would be adequate to accommodate the various pumping time restraints and longshore transport variability and still meet the 110,000 cy/yr bypassing objective.

Construction of the sand bypassing plant was completed and operations begun in January 1990. The principal components of the system include a crane-mounted jet pump operating from the south jetty fillet area, a pumphouse located adjacent to the south jetty, and the piping necessary to transport the bypassed sediment across the highway bridge to the north ocean beach. A more detailed discussion of the components and operation of the system is presented in Clausner et al (1991)

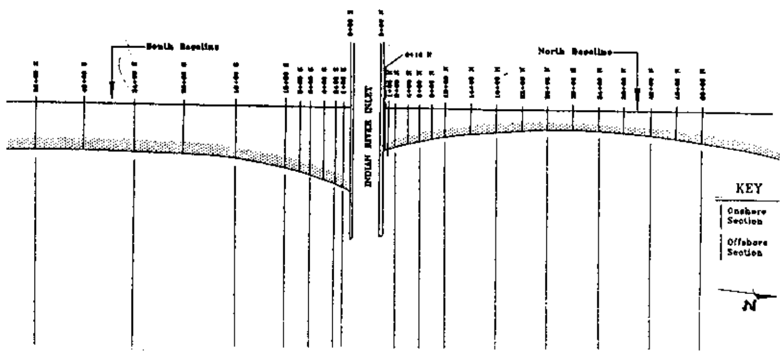
INDIAN RIVER INLET PRE- & POST-BYPASSING BEACH BEHAVIOR

Between November 1984 and September 1991 beach profile monitoring has been conducted approximately semi-annually on both the updrift and downdrift beaches adjacent to the inlet. The profiles have been collected at 28 locations spaced variably north and south of the inlet. These data provide a valuable insight into the beach behavior prior to the start of sand bypassing for comparison to beach response with the bypass plant in operation. This beach response information is critical to obtain optimum performance and operation of the system. Because both the north and south beaches are utilized for recreation, the bypass plant must be operated in a manner which achieves the stabilization of the north shore without inducing significant beach loss along the southern shoreline.

The survey area extends 5000 ft north of the north jetty centerline and 5000 ft south of the south jetty centerline. Of the 28 survey lines, 17 are located along the north beach at 100, 200, and 400 ft spacings with one line at the base of the north jetty. Eleven of the lines are located south of the inlet spaced 100, 200, and 800 ft apart. All lines are surveyed down to approximately -3.0 ft National Geodetic Vertical Datum (NGVD), but only 8 lines on the north beach and 7 lines on the south beach include the offshore portion of the profile. The locations and spacing of the profile lines are shown on Figure 3, which also indicates which survey lines include the offshore portion. Station numbers are the distance in hundreds of feet from the respective jetty centerline.

In order to quantify the pre- and post-bypassing beach behavior for comparison, profile lines at stations 2+00, 6+00, 10+00, 18+00, 34+00, and 50+00 on both the north and south beaches were chosen as representative of the different regimes of beach response near the inlet. The seasonal fluctuation of shoreline position and cumulative shoreline change are analyzed for both pre- and post-bypassing conditions. Beach width is defined as the distance from the baseline of each profile to the point of mean high water (MHW) on the beach face, approximately +2 ft NGVD. Seasonal fluctuations or adjustments are defined taken as the change in beach width between the semi-annual surveys. Survey dates typically correspond to the end of the summer months and end of the winter months.

Figure 3 : Indian River Inlet – North & South Ocean Beaches Monitoring Program Survey Lines



<u>Pre-Bypassing Shoreline Behavior:</u>

The pre-bypassing interval of surveys analyzed for this paper is from November 1984 though October 1989. The initial survey in November 1984 was obtained shortly after a large beach nourishment in which 468,000 cy of material was placed between stations 0+00 and 30+00. In the pre-bypassing surveys, there were no other beachfills within the survey area. The bypassing plant was brought on line in January 1990 during a north ocean beach nourishment operation of approximately 175,000 cubic yards obtained from the flood shoal and placed between stations 0+00 and 10+00.

North of the inlet the predominant longterm trend has been progressive retreat. The greatest net retreat, as measured from the Nov 1984 shoreline position of each profile, occurs at station 2+00, with the smallest net retreat at station 50+00. Figure 4 shows the relative shoreline position of profiles 2+00, 6+00 and 10+00 along the north beach from Nov 1984 through Sep 1991. Figure 5 shows the same information for profiles 18+00, 34+00 and 50+00 north. Within the first 2000 ft north of the north jetty there has been steady retreat with little summer accretion as would be expected in more stable beach locations. Between 2000 and 5000 ft north of the inlet there is also a longterm trend toward erosion. In contrast to the southern 2000 ft of shoreline, the northern 3000 ft of beach show seasonal variations in the profile.

In the first two years after the 1984 fill, the profiles from the jetty north to station 18+00 show steady shoreline retreat. This retreat reflects initial profile adjustment that often follows a beachfill. There were only a few small episodes of accretion within the zone 2000 ft north of the inlet during the pre-bypassing period. Figure 4 shows that within the first 1000 ft

north, except for one instance at station 10+00, the accretion occurred during the winter months when the net transport is in the southerly direction. It is assumed that this represents the trapping of some sediment within the shadow of the jetty during the winter months. At Station 18+00 Figure 5 shows a slight seasonal variation present with small episodes of acretion over the summer. In addition, by station 34+00 the effects of the inlet are less evident as large seasonal variations in shoreline positions occur, with a small net shoreline displacement from the initial positon. Station 50+00 shows the most stable profile of all the north beach profiles with the smallest net shoreline change.

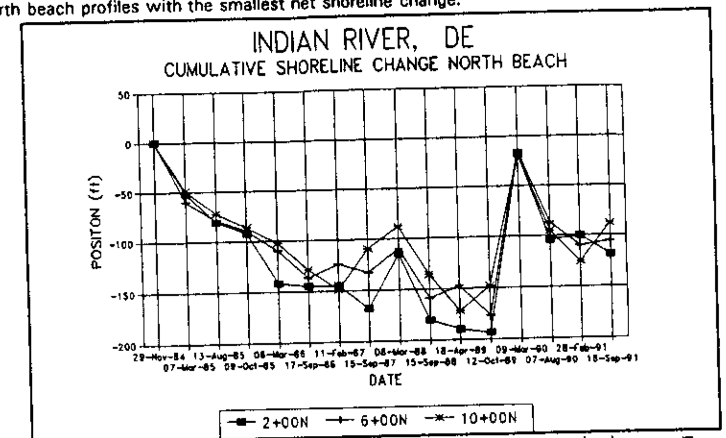


Figure 4: Cumulative Shoreline Position For Stations 2+00, 6+00 & 10+00 North.

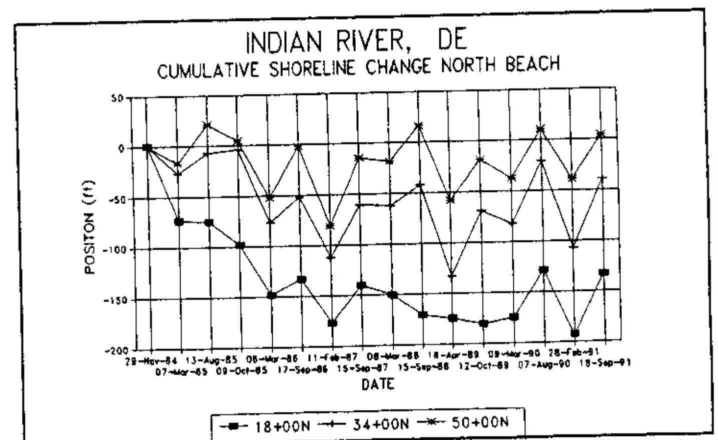


Figure 5: Cumulative Shoreline Position for Stations 18+00, 34+00, & 50+00 North.

South of the inlet the longterm trend in beach width has been one of overall stability with a small tendency towards accretion. The further away from the inlet the station is located, the smaller the overall observed variation in beach width. Within the first 1000 ft south of the jetty there are large variations in beach width between successive survey intervals. This can be seen on Figure 6, which presents the relative shoreline position of profiles 2+00, 6+00, & 10+00 south from Nov 1984 through Sep 1991. Figure 7 shows the same data for profiles 18+00, 34+00, and 50+00 south of the inlet. It can be seen that south of station 10+00 the shoreline appears to be more stable. There are seasonal variations

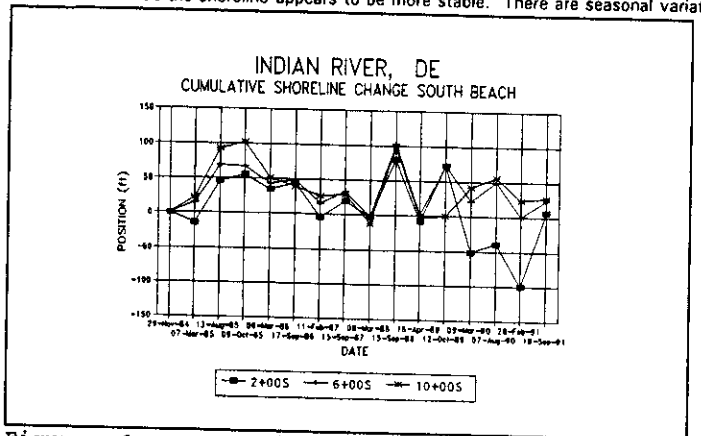


Figure 6: Cumulative Shoreline Position for Stations 2+00, 6+00, & 10+00 South.

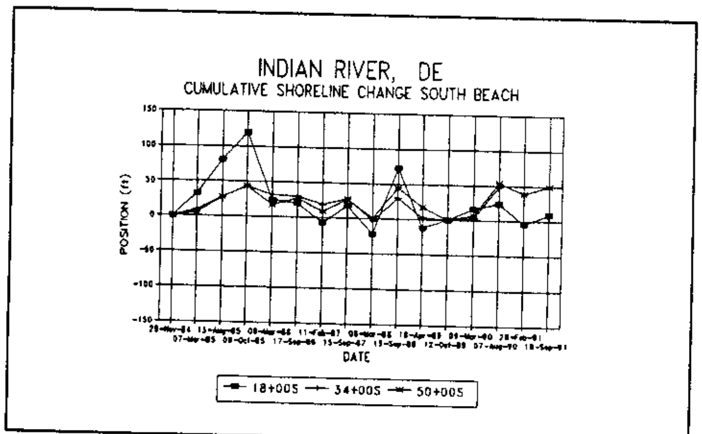


Figure 7: Cumulative Shoreline Position for Stations 18+00, 34+00, & 50+00 South.

in beach width, but the range is smaller than is seen within 1000 ft of the inlet.

Table 1 quantifies the pre-bypassing beach width seasonal fluctuations as well as the net changes in shoreline position. The north beach shows the trend of having the greatest net shoreline change close to the north jetty at station 2+00N (-194 ft), and reducing at each station northward to 50+00N (-19 ft). It is of note that at any time during the pre-bypassing period only station 50+00N had a beachwidth greater than its 1984 width. However, station 18+00N actually shows the second largest net shoreline retreat of -181 ft over the pre-bypassing period.

The fact that the highest values of retreat are observed within 2000 ft of the north jetty is consistent with longterm history, showing the need for repeated beach nourishment in this area to protect the coastal highway. It should be noted that the largest measured recession within the first 2000 ft occurred immediately after the Nov 1984 beachfill, reflecting initial profile adjustment that is typical just after a beachfill. By 3000 ft north the effects of the of the inlet become less apparent as can be seen by the reduced amount of net shoreline change at station 34+00N (-69 ft) and 50+00N (-19 ft), and smaller average relative beachwidths of -59 ft and -18 ft, respectively, compared to the average relative beachwidths of stations 2+00N (-137 ft) and 10+00 (-113 ft).

TABLE 1. PRE-BYPASSING SHORELINE FLUCTUATIONS (ft).

PROFILE LOCATION	WINTER INTERVAL ADJUSTMENT RANGE			SUMMER INTERVAL ADJUSTMENT RANGE			SHORE POSITION CHANGE	SHORELINE POSITION RELATIVE TO 11/84		
	MAX	MIN	AVE	MAX	MIN	AVE	11/84 TO 10/89	MAX	MIN	AVE
50+00 N	-79	-4	-47	66	35	45	-19	21	-81	-18
34+00 N	-91	-1	-50	64	20	36	-69	-4	-133	-59
18+00 N	-73	-4	-36	37	-21	5	-181	-73	-181	-138
10+00 N	-49	21	-20	40	-48	-7	-148	-49	-172	-113
6+00 N	-60	23	-6	-9	-49	-27	-177	-60	-177	-121
2+00 N	-52	53	-12	-3	-66	-25	-194	-52	-194	-137
2+00 S	-88	-14	-39	83	10	51	72	80	-14	23
6+00 S	-100	15	-36	105	6	50	70	103	-3	35
10+00 S	-100	22	-37	109	-12	43	40'	102	-12	40
18+00 S	-96	32	-43	95	-4	41	30'	120	-22	30
34+00 S	-31	9	-14	33	-2	15	30'	43	-3	20
50+00 S	-27	5	-18	44	10	24	42'	45	1	20

' = Estimated values based on ave. pre-bypassing change at each profile and the recorded 10/89 2+00 & 6+00 So. profile accretion.
N = NORTH BEACH S = SOUTH BEACH - = RETREAT.

South of the inlet the beach appears to have a small longterm accretion trend, but at a much lower rate than the north beach is eroding. Table 1 indicates that the greatest seasonal changes occur within the first 2000 ft of the south jetty, with the maximum in this area on the order of 100 ft, and the average adjustments around 50 ft. Stations 6+00S and 10+00S experienced the greatest maximum seasonal adjustments. The maximum observed winter and summer interval adjustments for station 6+00S were -100 ft and +105 ft, respectively, and for

10+00S they were -100 ft and +109 ft. Stations 34+00 and 50+00 had the lowest average seasonal adjustments, and the lowest range of relative shoreline position. The low amount of maximum seasonal adjustment and low net shoreline change appear to show that by station 34+00S the beach is fairly stable and is not greatly affected by the complicated inlet processes.

Post-Bypassing Shoreline Behavior:

Figures 4 through 7 also include the measured change in beach width at the selected stations for the last four survey intervals since the bypassing plant started operation (Mar 90, Aug 90, Feb 91, and Sep 91). During the last survey interval, Feb 91 to Sep 91, the plant did not operate during the months of June through August. The north beach was nourished with 175,000 cy placed within the first 1200 ft north of the inlet during Jan - Mar 90 that has affected beach width change. This nourished section of shoreline exhibited the expected increase in beach width, which averaged approximately 170 ft. As was the case after the 1984 fill, the stations which received the fill show large initial profile adjustments in the first interval after bypassing was started. This can be seen on Figure 4 which shows Stations 2+00, 6+00, and 10+00 N retreating 86 ft, 72 ft, and 77 ft respectively, between Mar 90 and Aug 90, despite the bypassing of approximately 52,000 cy of material to between 200 and 600 feet north of the inlet. Figure 5 shows that further north of the fill area the beach at stations 34+00 and 50+00 N show little effect of the bypassing or beachfill in this initial survey interval, continuing to adjust as was observed during the pre-bypassing period. However station 18+00N does show a large summer interval accretion of 47 ft, which is larger than its pre-bypassing maximum. It is probable that this station showed effect from the beach fill and bypassing sediment migrating northward during the summer. In the interval between Aug 90 and Feb 91 approximately 48,000 cy of material was bypassed onto the north beach and approximately 36,000 cy was bypassed in the last survey interval Feb 91 to Sep 91. It can be seen on Figure 4 that for the last two intervals stations 2+00, 6+00 and 10+00 appear to be stabilizing. Both station 6+000 and 10+00 showed beach width increases between Feb 91 and Sep 91, this is the first summer interval since monitoring began that station 6+00 did not show a reduction in beach width, and the gain at 10+00N of 38 ft is the second largest observed at that profile. In the last two intervals station 18+00 N continued to show a substantial seasonal variation in beach width with a small net positive change similar to stations 34+00 and 50+00N. In fact in the last survey interval 18+00N had a gain in beach width of 60 ft, which was the largest gain observed at this station.

Data in Figure 7 suggest that south of the inlet at stations 18+00, 34+00, and 50+00 S the bypassing operation has had no significant effect on the beach width, with stations 50+00 and 34+00 both exceeding their maximum pre-bypassing beachwidths each summer interval since bypassing started. South of the inlet adjacent to the location where the fillet is being mined, station 2+00S, changes in beach width have been larger than pre-bypassing changes. During the first interval the beach retreated -123 ft which is 35 ft greater than the previous maximum of -88 ft reported in Table 1. During the next interval which was the summer of 1990, Mar 90 to Aug 90, the plant operated all summer and mined the largest quantity of material. Subsequently the beach recovered only very slightly, +13 ft. This value was the second smallest summer recovery observed and well below the average of 50 ft. In the next interval station 2+00 retreated 60 ft which is within the historic winter shoreline adjustment. Station 2+00 recovered during the last interval, in which the plant was not operated between June and August, increasing beach width by 107 ft. This is the largest increase in beach width observed at this station. Figure 6 also shows the results of post-bypassing at stations 6+00 and 10+00 S. They both exhibit similar trends to station 2+00, with changes in beach width within the range of pre-bypassing changes. But during the summer interval Mar 90 to Aug 90 they did not recover as much as would be expected. Station 6+00 S accreted only 30 ft which is in the lower range of the historical summer interval change. This marks the only time that station 6+00 S has accreted less than stations 34+00 & 50+00S. However, in the last summer interval 6+00S has again experienced a larger accretion than stations 34+00 & 50+00S.

Based on the initial results of monitoring the Indian River north and south beaches, it appears that the bypass plant has started to have its intended effect of stabilization of the north

shoreline. Within the first 1000 ft north of the jetty, after the initial profile adjustment to the 1990 nourishment, the beach appears to have reversed its trend of steady erosion and is actually showing small seasonal variations of accretion and erosion while maintaining a relatively stable beach width. Further north of the inlet at stations 34+00 and 50+00N the shoreline is showing a net accretion over the bypassing period while showing large seasonal changes in beach width, accreting during the summer and eroding in the winter.

South of the inlet the initial results indicate that bypassing has probably reduced the beach width in the zone up to 600 ft south of the inlet. The reduction appears to be the result of bypassing too large a quantity of material through the summer months of the interval Mar 90 through Aug 90 when the predominant northward transport of sediment is renourishing the fillet area. During the last interval Feb 91 through Sep 91 in which bypassing did not occur over the summer months, the beach widths in the vicinity of the bypass plant accreted more than the historical summer interval maximum. Bypassing during the winter months at the levels so far bypassed has not shown increased retreat in beach width over the pre-bypassing levels. It is noted that though it appears the shoreline adjacent to the bypass operation is slightly narrower over the first 600 ft south of the inlet than it would most likely be without bypassing, it is still over 175 ft wide from the base of the dune to the high tide line. The zone south of Station 6+00 S has remained relatively unaffected by the bypassing. All observed post bypassing interval changes have been within their respective historic ranges, and stations 34+00 and 50+00 S have shown substantial net increase in beach width. It appears that by not operating the bypass plant during the summer months it may be possible to achieve the goal of stabilizing the north beach while not significantly reducing width of the south beach.

The long term reactions of the beaches north and south of the inlet to the bypassing are not easy to predict after only three survey intervals after the initial nourishment of the north beach. While the mechanical and hydraulic portions of the bypass system are capable of bypassing in excess of the 110,000 cy/yr desired, the controlling point of the system is that the amount of sediment delivered to the system can vary significantly from year to year. If the plant bypasses 100,000 cy in a year in which only 75,000 or 50,000 cy are transported to the system, the south beach will obviously suffer a reduction in beach width. The continued monitoring and analysis of the bypass data will enable better management to optimize the future bypassing rates and locations to achieve the goal of stabilizing the north beach while minimizing the reduction of beach width adjacent to the bypassing operation.

GENERALIZED INLET SCOUR

From the date of completion of jetty construction and initial channel excavation in 1939 until the mid-1970 period, the intra-jetty channel area exhibited a modest rate of deepening which was not perceived as a problem, especially in light of the highly visible interior shoreline and north ocean beach erosion problems already discussed. This deepening can be characterized by comparing the mean 1942 inlet depth in the 1500 ft long zone between the jetties, about 10 ft MLW, with the comparable value in 1974 of about 25 ft MLW. Although there were localized areas of greater and lesser depths in 1974, the significant point is that the scour was not confined to any particular location - the entire intra-jetty zone was deepening. Despite the deepening, there were no damages attributable exclusively to channel scour. The loss of about 150 lineal ft of the seaward end of the north jetty by this time was attributed to the combined effects of scour and wave exposure, but had not impaired the function of the jetties to provide a suitable navigation channel.

In 1979, the Philadelphia District initiated contact with the Delaware Department of Transportation (DOT) regarding channel scour in the vicinity of the two bridge piers in the inlet. Channel deepening around the north pier in particular had progressed to the point that the steel sheet pile cofferdam around the bridge bearing piles was in danger of being undermined. Then in 1986, the District initiated coordination with CERC to discuss possible studies needed to determine why the generalized scour was occurring, why the scour had apparently accelerated in the mid-70's, and to determine how much additional deepening was likely before the inlet geometry attained some degree of equilibrium.

A study plan was prepared by CERC and accepted and funded by the District in 1988. A principal feature of the plan included the development of a 2-D hydrodynamic model of the ocean-inlet-bays system, including the necessary prototype data collection to calibrate and verify the model. The WES Implicit Flooding Model (WIFM) was selected for this effort. The model test plan utilized a 63-hour prototype spring tide hydrograph for the ocean boundary condition, and ran this hydrograph against a series of inlet and bay geometries representative of various stages of inlet evolution, from the immediate post-jetty construction condition to that existing in 1988. Stages and velocities were computed at the inlet throat as well as at a number of interior bay locations for which historic prototype data were available. Additionally, several inlet geometries were run which simulated continued scour of the intra-jetty area by as much as 20 ft beyond than the 1988 conditions.

The results of the model simulations show that the progressive enlargement of the inlet channel is accompanied by a persistent increase in the tidal discharge (i.e., tidal prism) and by corresponding increases in tide range for the various interior bay locations. Further, the model runs do not show any tendency for the tidal discharge, inlet velocities, or bay tide ranges to approach equilibrium values even with the 20 ft inlet deepening scenario. If the model results are accurate, then significant additional enlargement of the inlet may occur before hydraulic equilibrium of the ocean-inlet-bays system is attained.

The CERC study also attempted to assess possible contributing factors to the accelerated scour which began in the mid-1970's. A number of events occurred in the same period as the onset of accelerated scour, including the removal of the old (pre-1965) bridge pier and pile network across the inlet, the beginning of use of the inlet flood shoal as a source for north ocean beach fill, and the progression of channel scour through the approximately 30 ft thick surface layer of sandy sediment into the underlying stratum of older, estuarine "mud". Much of the channel bottom presently consists of the exposed older clay layer, and the potential for significant additional scour is considered likely. In order to minimize the risk of further scour around the bridge piers, the State DOT constructed a $2.7M scour protection project in 1989 utilizing a placed stone blanket. The armored channel in the vicinity of the bridge piers has been stable since 1989. The present inlet configuration is shown in Figure 8 which is a plot of the June 1991 hydrographic survey contoured at 10 ft depth intervals.

CONCLUSIONS

In its natural state, Indian River Inlet presented a number of problems for residents of coastal Delaware. It was an ephemeral feature of the coastline, with a natural tendency to migrate along the shoreline and frequently close due to an excess of littoral sediments delivered to the channel mouth. It did not offer a reliable navigation passage from the interior bays to the ocean, and the frequent inlet closures led to unacceptable bay water quality.

Construction of jetties and initial channel dredging in 1938-39 solved the problems of navigability and bay water quality, but initiated a sequence of changes in the inlet vicinity which have presented a number of coastal engineering challenges over the past 50 years, at least one of which persists to the present.

The interior shoreline erosion problem has been resolved through application of rather simple and conventional, if not cheap, coastal engineering technology - stone revetments.

The north ocean beach erosion problem was initially mitigated over a 30 (+) year period by the placement of dredged sediments from back bay and flood shoal borrow sources. In 1990, a mobile jet pump sand bypassing system was completed. This system represents something of a departure from conventional coastal engineering technology. The system components have proven capable of transporting the quantities of sediment for which they were designed. It will probably take more than the presently elapsed 1.8 years to know if the objectives of a stabilized north beach and minimally affected south beach have been met, but early results are favorable.

The most recent of the coastal engineering problems to be recognized at Indian River Inlet - generalized channel scour - continues unabated except at the location of the bridge piers. Investigations are presently underway to better quantify the potential risk posed to the stability of the jetties, and to develop a preliminary range of coastal engineering solutions to this scour. It would appear that the only ocean inlet in one of our smallest states has proven to be far more problematic than anticipated by some of the better-known coastal engineers practicing in the 1930's. Experience with Indian River Inlet over the past 57 years should hold some valuable lessons for coastal engineers of the 1990's and beyond.

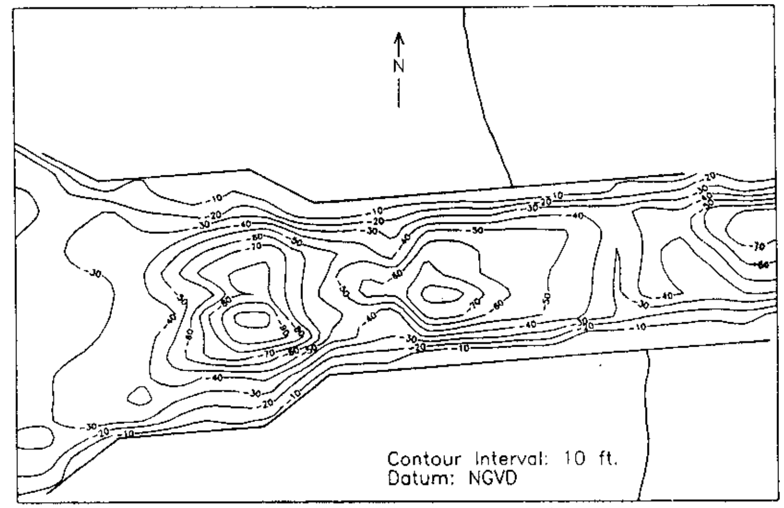


Figure 8. Hydrographic Survey 6/91

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