CHAPTER 24

BY-PASSING AND BACKPASSING WITH SPECIAL REFERENCE TO CONDITIONS IN FLORIDA

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ABSTRACT

By-passing by natural action is mentioned with special reference to Florida Inlets and to some other inlets in the United States and abroad. Natural by-passing at harbors on open shores is dealt with briefly. Present status of by-passing plant operations in Florida is reviewed. Inasmuch as it is evident that by-passing plants - partly because of the tidal flow which discharges material in the ocean and in the bay and partly as a result of the rise of sea level - will not be able to solve more than a certain part of a beach erosion problem--replenishment by sand from other sources is indispensable. The most logical source is offshore deposits. Material may be brought to shore by "backpassing" using an offshore scraper (useful for maintenance) or by a special hydraulic dredge (for major improvements). If the borrow area is located close to shore, the question arises of whether the borrow pit will fill up again by material from further offshore, material from the sides or from material dragged out by waves from the beach. The report describes briefly tests on Jupiter Island using an offshore scraper. The success of this operation is checked by fluorescent tracers placed on the beach and on all sides of the borrow pit.

NATURAL BY-PASSING AT COASTAL INLETS

GENERAL

Natural by-passing at several inlets in and outside the United States is dealt with by P. Bruun and F. Gerritsen (3) and (4). Most inlets by-pass material partly by tidal flow action and partly by transfer of sand from one side of the inlet to the other on a shoal or offshore bar. Figure 1 shows a normal bottom profile without an inlet channel. The profile carries net M^3/year longshore. Figure 2 demonstrates the changes which occur in a bottom profile when a breakthrough has taken place and the drift mainly takes place on a bar across the inlet entrance.

It will be of interest to consider the sand drift budget at the inlet. If the total amount of material carried to the inlet from all sides is $M_t = M_{total}$ and p per cent is transferred by inlet flow, $(1 - p) M_t$ must be by-passed on a bar or shoal (Figure 3). The inlet currents carry bottom material for and back in the inlet. If an equilibrium condition develops inlet currents are able to push the "surplus material" which entered in the inlet from the sides out of the inlet channel for depositing offshore or in the bay or for further migration on the ocean bottom. The dimensionless parameter M_{net}/Q_{max} seems to be of significance for the magnitude of by-passing. The value of this ratio indicates whether by-passing is a predominately "bar" or a predominately "tidal flow transfer." By the latter material is

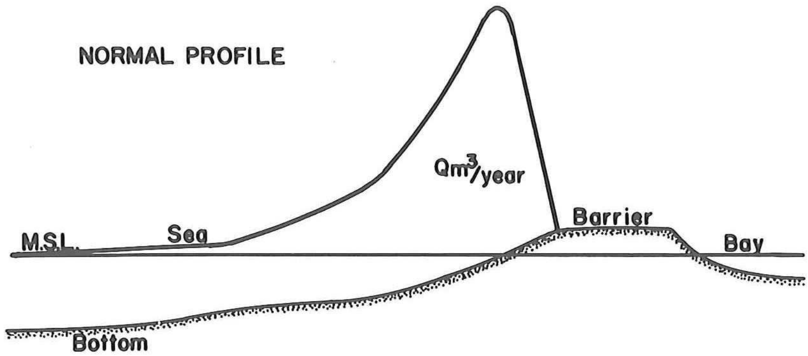

Figure 1. Littoral Drift (Mm^3/year) in a Normal Bottom Profile (Bruun and Gerritsen, 1961)

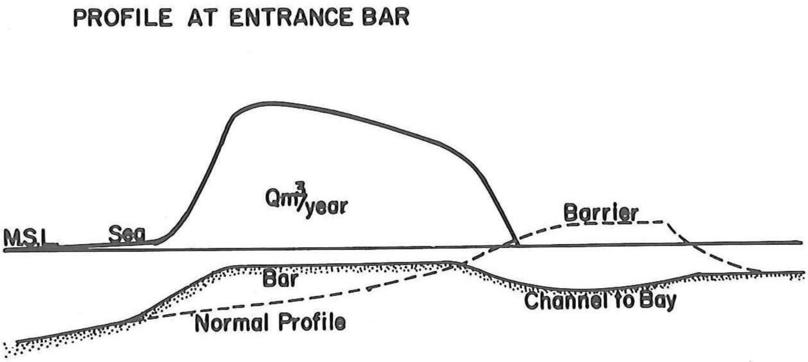

Figure 2. Change of Littoral Drift Pattern caused by Break-through of an Inlet (Bruun and Gerritsen, 1961)

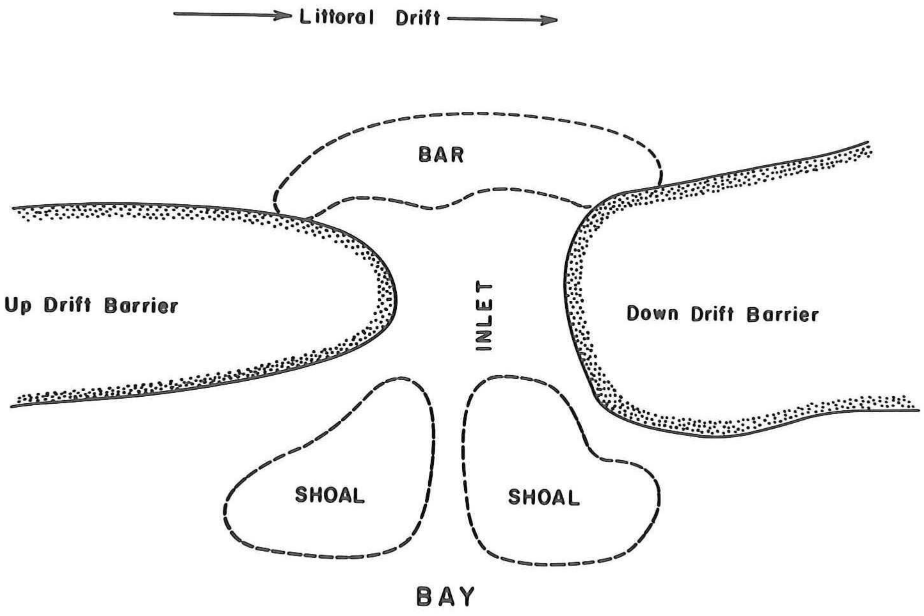

Figure 3. Coastal Inlet with a Predominant Bar By-Passing (Bruun and Gerritsen, 1961)

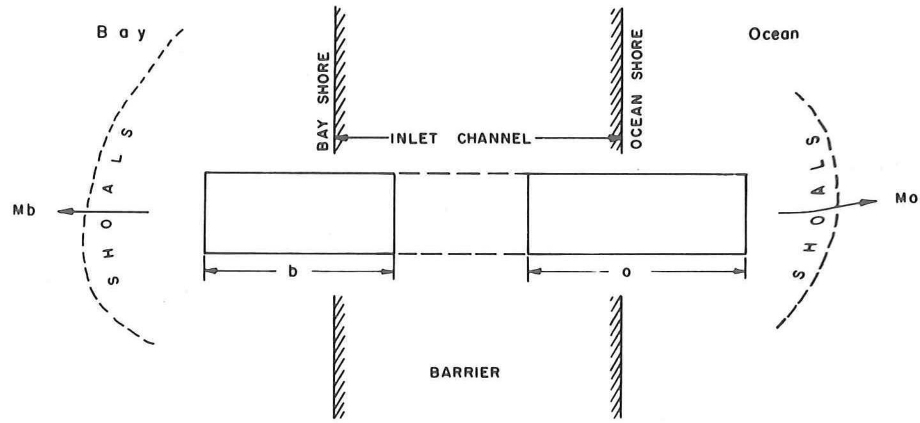

Figure 4. "Rolling Carpet" of Bottom Material in Inlet Channel

flushed out of the inlet by ebb currents carrying the material away from the inlet entrance to the offshore area and possibly in downdrift direction.

Reference is made to Table 1 (Reference 4). When Q_{max} is expressed in cub.yds/sec. and M_{net} in cub.yds/yr., a value of $\frac{M_{net}}{Q_{max}} = r$ between 5 and 900 has been found for the inlets considered.

From practical experience about by-passing, the following rule may be used as a guide:

$r < 10 - 20$ indicates predominant tidal flow by-passing (little or no bar formation)

$r > 200 - 300$ indicates predominate bar by-passing with typical bar or shoal formation

That M_{net} is small compared to Q_{max} does not necessarily mean that conditions are ideal for tidal flow by-passing. A large Q_{max}, and therefore a smaller $\frac{M_{net}}{Q_{max}}$, may still mean unsatisfactory by-passing of material if the tidal flow is not utilized properly for flushing of material to downdrift beaches. Instead of being carried in a downdrift direction the material may be jetted out in deep water and settle there in a shoal (Figures 7 and 9).

In the inlet channel bottom material is moving in both directions by the flood and ebb currents which in case of no fresh water flow and a large tidal prism may be rather symmetrical. In the initial stage of the development when the inlet channel may be short and the inlet cross section is expanding the situation is as depicted in Figure 4, showing a longitudinal sector in the inlet.

The sheet-layer (bed load) motion may be compared to the motion of "rolling carpets," lengths b (bay) and o (ocean). By the movement of these carpets, part of these is lost on sea shoals ($M_o = M_{ocean}$) and another part on bay shoals ($M_b = M_{bay}$). If no material at all is transferred to the inlet channel by littoral drift from both sides, the channel will gradually deepen and widen until it becomes non-scouring. If M_b plus $M_o = V$ = the flushing ability of the inlet equals the amount of drift to the inlet from the sides an equilibrium condition exists although it is not everlasting because of the continuation of deposits at both ends of the channel. If the inlet channel grows very long, a situation may develop by which the inlet current gradually weakens and the cross section area of the inlet gorge decreases simultaneously because of decreases in tidal prism. This may finally result in lack of ability of the inlet channel to flush itself adequately for all the material brought to the inlet from the longshore drift. Considering the all-over stability it seems that one is faced with the following three cases:

TABLE 1
Flow and Littoral Drift Characteristics (Reference 4)

Inlet (1)	Q_{max} in cub yds per sec (Springtide Conditions) (2)	M_{net} in cub yds per yr (Order of Magnitude) (3)	$r = \frac{M_{net}}{Q_{max}}$ (4)
Amelandse Gat, Holland	36,000	10^6	28
Aveiro, Portugal	9,000[1]	$>10^6$	111
Big Pass, Fla.	720	10^5	139
Brielse Maas, Holland (before closing)	2,740	10^6	365
Brouwershavense Gat, Holland	30,000	10^6	33
Calcasieu Pass, La.	2,600	10^5	38
East Pass, Fla.	1,720	10^5	58
Eyerlandse Gat, Holland	19,000	10^6	53
Figueira Da Foz, Portugal	1,100	$>10^6$	910
Fort Pierce Inlet, Fla.	3,700	$1/4\ 10^6$	68
Gasparilla Pass, Fla.	910	10^5	110
Grays Harbor, Oreg.	48,000	10^6	21
Harlingvliet, Holland	25,000	10^6	40
Inlet of Texel, Holland	97,000	10^6	10
Inlet of Vlie, Holland	94,000	10^6	11
Longboat Pass, Fla.	1,430	10^5	70
Mission Bay, Calif. (before dredging)	1,130	10^5	88
Oosterschelde, Holland	100,000	10^6	10
Oregon Inlet, N. C.	5,100[1]	$3/4\ 10^6$	147
Ponce de Leon Inlet, Fla.	1,450	$1/2\ 10^6$	345
Port Aransas, Tex.	1,870	10^5	54
St. Augustine Inlet, Fla.	2,700	$1/2\ 10^6$	175
San Francisco, Calif.	210,000	10^6	5
Scheveningen, Holland	sluices	$3/4\ 10^6$	---
Thorsminde, Denmark	sluices	$1/2\ 10^6$	---
Thyboron, Denmark	7,450	10^6	134
White Sands, Denmark	sluices	$1/2\ 10^6$	---
Westerscheide, Holland	115,000	10^6	9

[1]increasing

1. Short channel: $V > pM_{total}$. This will cause an unstable condition. The inlet is widening and probably lengthening. The situation may develop towards a "non-scouring" channel as discussed in a following paragraph on inlet stability.

2. Medium channel length: $V = pM_{total}$. This will result in a stable channel as long as $V = pM$ is valid.

3. Long channel: $V < pM_{total}$. This will result in an unstable condition. The inlet is shoaling because material is pouring in the inlet channel from both sides and inlet currents are not able to flush the material out. It may also happen that V increases to a maximum capacity, but if V still is less than pM_{total} the channel will again start decreasing its flushing ability at the same time as a bar or shoal may develop at the ocean entrance of the inlet. The ability to transfer material over the bar may then increase until a stationary condition develops by which $(1-p)\ M_{total}$ is transferred over the bar, while pM_{total} is flushed out on both sides of the inlet channel for depositing on shoals, or it is perhaps mainly flushed out on one side, that is, on the ocean side if the ebb current is the strongest (which usually is the case). If p is relatively small and the tidal prism is large enough to meet temporary increases of p during extreme storms an equilibrium condition may result which may preserve the inlet as a tidal channel for centuries. See the sections below on "Florida Inlets" and on "Tidal Inlets Stability Considerations."

The quantity of littoral material pouring into the inlet from the adjacent shores depends upon many partly interrelated factors including the longshore component of the wave energy, the geometrical shape of the beach and bottom profile, the shore line geometry, and material characteristics. There is, however, another important factor which is the availability of material. It is known that coastal protection structures, whether groins or certain types of sea walls, slow down the quantity of littoral drift. Inlets may sometimes cause severe decreases of the littoral drift for some distance or for several miles downdrift. If the littoral drift is strong, and the tidal prism is less, more breakthroughs may occur and they may stay open for a longer period of time.

The United States East Coast includes an almost continuous barrier coast with numerous inlets some of which have stayed open as long as they have been known. Others have opened and closed continuously. The tendency toward breakthroughs is usually increasing in downdrift (usually south) direction, simply because the littoral drift decreases with the number of inlets accompanied by sea and bay shoals upon which material deposits temporarily or permanently. The North Carolina Shore, north of Cape Hatteras has at present only one inlet (The Oregon Inlet), but others have existed. The net south littoral drift is probably above 500,000 cub.yds/year. Occasionally new inlets have broken through and closed again very shortly.

Oregon Inlet is depicted on English maps from the 16th Century, but other historical sources indicate that the present inlet was opened by a seiche generated in the Pamlico Sound during the passage of a tropical storm in September 1846. In the period 1846-1952, this inlet migrated 1.5 to 2 miles southward.

Maintenance by dredging the last 3 to 4 years has been about 100,000 cub.yds/year. The ocean bar channel has authorized project dimensions of 400 ft. width and 14 ft. depth, but shoals 8 to 9 ft. depth occur. Eight to nine miles south of the present location of Oregon Inlet, another inlet was located, possibly for centuries. It did however close in January 1922 and was re-opened in 1924 as "New Inlet," but closed again in the 1930's. In 1962, the March 9 to 11th storm opened up a new inlet just north of Buxton (Village of Cape Hatteras). This inlet was closed by a hydraulic dredge the following year. A withdrawn dyke would have prohibited that kind of costly accident.

West of Cape Hatteras, the littoral drift is undoubtedly of much less magnitude. The first inlet is Hatteras Inlet which was found open in 1585 and has remained open since then. The inlet migrated about 3,600 ft. southwest between 1852 and 1905 and has later been rather stable in location. With its 50,000 sq. ft. gorge cross sectional area the Hatteras Inlet has swallowed huge quantities of sand. No wonder, therefore, that the next island, the Ocracokee Island, has suffered very severe erosion by which all dunes have been washed out in the northern part. The situation at the southern part of the Ocracokee Island is similar. Since 1830, the spit has extended about 8,000 ft. in the southwesterly direction in the next inlet, the Ocracokee Inlet and huge quantities of material have accumulated in shoals thereby depriving downdrift beaches.

The shore from here on down to Cape Lookout consists of washout barriers and inlets causing continuous drain of material from the shore for depositing in shoals. Many inlets however have not been able to stay open because of overwhelming littoral transport to the inlet entrance compared to the available tidal prism. The shore between Cape Henry and Cape Lookout (about 200 miles) today has only 3 open inlets but 10 to 12 "fossil" inlets which have been open at various times.

The California shore has only a few rather short barriers but is blessed with one of the largest tidal inlets in existence. The San Francisco Harbor has a tidal prism of 2,880 sq. miles times ft. flowing through the Golden Gate (875,000 sq. ft.). The r (M_{net}/Q_{max}) value is 5 (Table 1). The littoral drift is not very predominant, but strong tidal currents and heavy wave action together with material from the bay and the north shore are responsible for the huge half-moon shaped offshore bar with depths of from 12 to 18 ft. (Figure 5). Some littoral drift material passes across the (now 50 ft.) deep navigation channel nourishing the beaches south of Cliff House on the San Francisco side. Studies of the distribution of grain sizes and of heavy minerals in the bar and inlet area as described by Byron Schatz

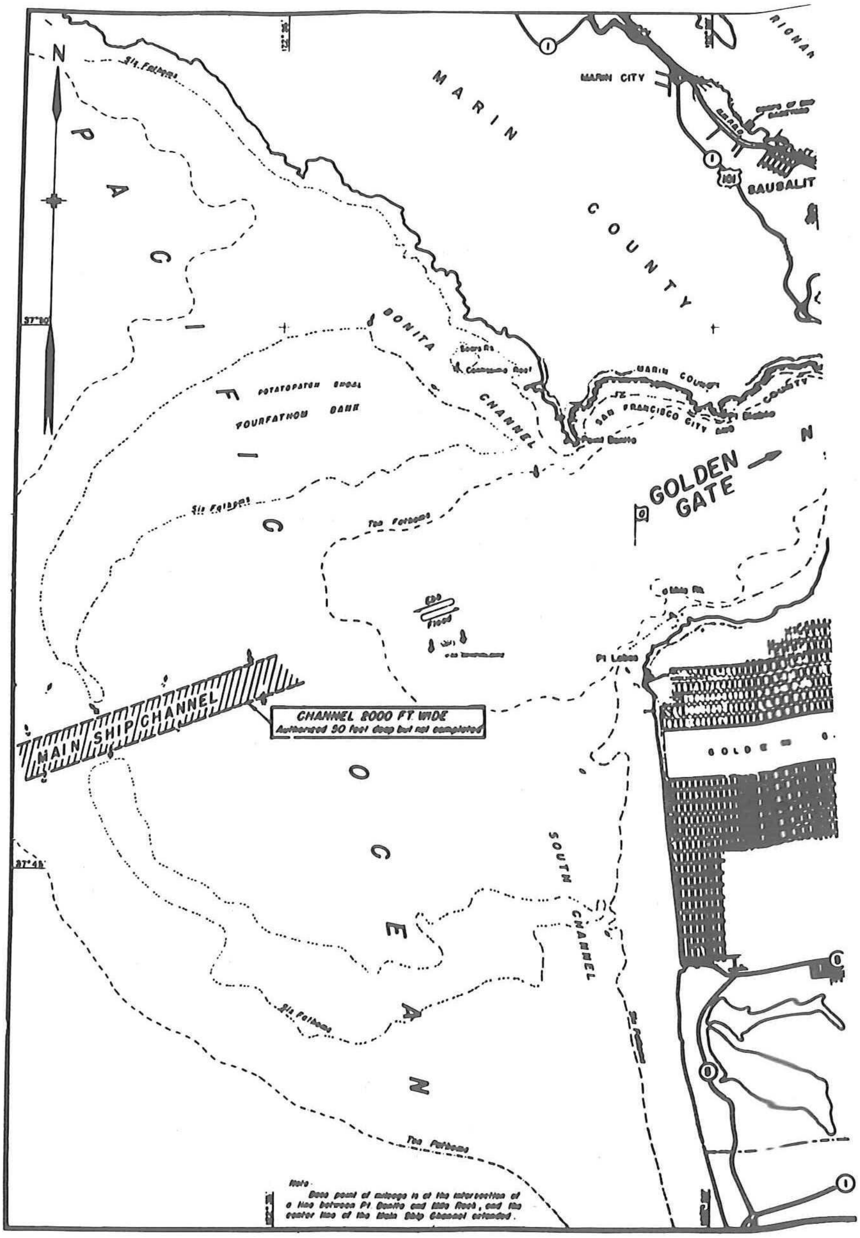

Figure 5. The Golden Gate and its Ocean Shoals (U. S. Army Corps of Engineers Annual Report)

and others (15) demonstrated clearly that grain size diameters rather than heavy minerals may be helpful in determining the direction of sand transport.

Going for a visit abroad, a small but interesting case has been in operation at Thorsminde (Thors Inlet) on the Danish North Sea Coast (Table 1). Fig. 6 (a) and Fig. 6 (b) are charts of the inlet, which has navigation locks as well as locks for general flow. Tidal range is approximately 1 ft. and wave action often heavy with up to 12 to 15 ft. waves. Littoral drift is estimated to be of the order 1/2 million to 1 million cub.yds/yr. The inlet is protected by two jetties. Until 1944 both were 500 ft. long (Reference 4). The following years the north jetty was extended 200 ft. and during the period 1942 - 1947 two groins were built on the updrift side of the inlet. The groins should catch excessive amounts of material migrating toward the inlet and the jetty extension should cause material to be by-passed farther seaward. Fig. 6 (a) shows the situation on June 23, 1941, when the inlet conditions were particularly bad, with depths less than 3 ft. between the jetties (normally 6 ft. to 7 ft.). It can be seen that there is no bar in front of the inlet. Fig. 6 (b) shows the conditions on November 7 to 11, 1942. There is a bar, and at the same time the inlet conditions are good with depth of about 8 ft. between the jetties. The problem of shoaling which always takes place when the bar disappears, may be explained by the different distribution of littoral drift in a profile with a bar and a profile without a bar. At the first mentioned profile, much sand by-passes on the bar where waves break. At the other profile most material migrates close to the shore line causing rapid shoaling as soon as it meets an obstruction, as for example, the inlet entrance. During and after World War II the north jetty was extended about 130 ft. (40m). The relief was of temporary nature only and further improvements became necessary. In 1958/1959 a 550 ft. (160m) jetty was erected about 500 ft. (150m) on the updrift side of the inlet. Before this improvement started, the depth on the bar varied from about 6 to approximately 8 ft. The jetty, however, is apparently located too far from the entrance and so far the improvements are not satisfactory because the bar continues to "creep around" the jetty.

A similar situation exists at another inlet provided with sluices and 1,000 ft. long entrance jetties and located about 50 miles south of Thorsminde at Hvide Sande (White Sands). Model experiments were conducted on this installation in the 1920's and was used for design of the configuration of the entrance area and the jetties. The depth on the entrance bar was usually 10 to 12 ft., but less after storms.

The erection of a sand trap jetty was started in 1961. This 600m long jetty was located 100m north of the entrance. Before construction started the entrance channel had to pass over an offshore bar with depths of about 2.5m. The jetty pushed the bar somewhat seaward and upon completion of the jetty in 1963 depth was greater than 5m over the bar while depth of about 4m occurred in the entrance channel inside the bar. In 1964 shoals had reformed with depth down to 3.1 - 3.3m

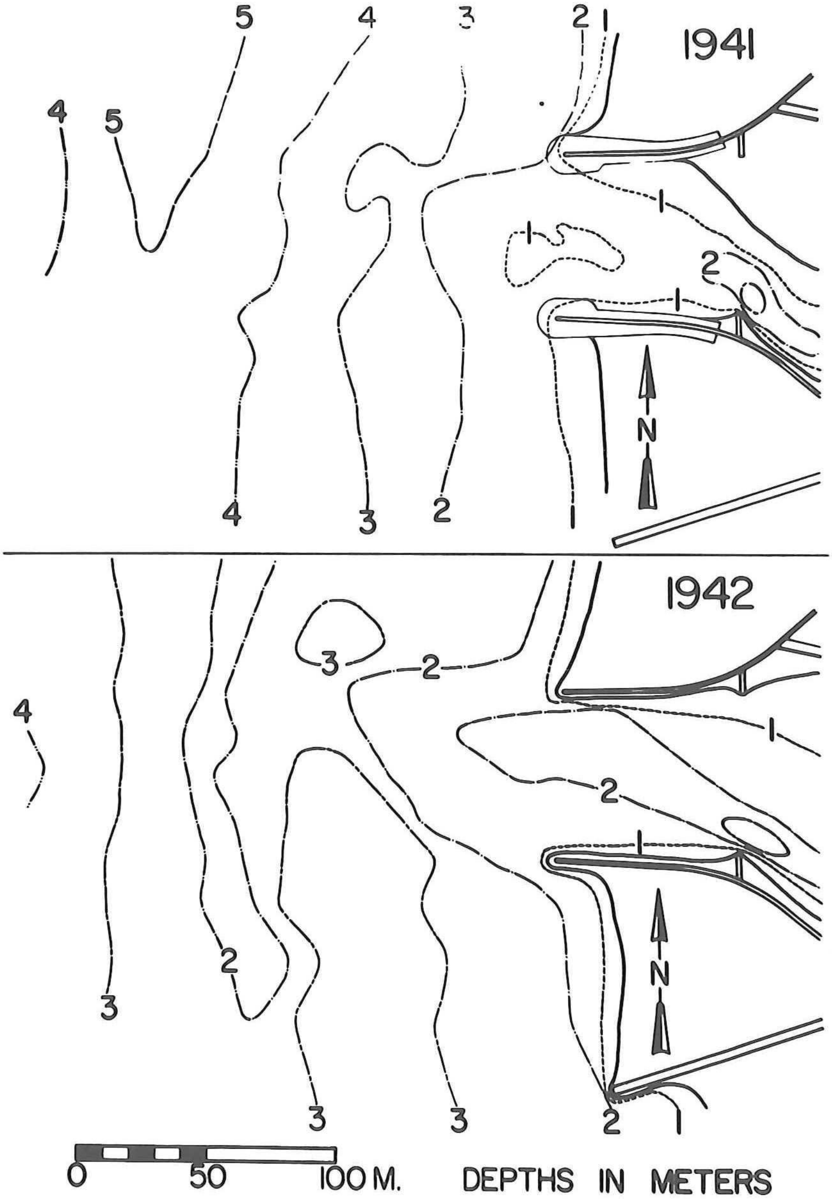

Figure 6. Thorsminde Inlet, Danish North Sea Coast

in the entrance area outside the sand trap jetty and the condition was similar in 1965. The improvement therefore has not been too great when compared to 1961 but navigation is still greatly improved for NW storms.

It is apparent from these two cases that by-passing may occur satisfactorily and without giving rise to lee-side erosion, on a bar of a certain depth and width. However, irregularities in the amount of littoral drift may momentarily result in shoaling of the respective inlets that must be cleared by flushing by operation of their sluice gates and/or by dredging.

FLORIDA INLETS

Considering Florida Inlets (Reference 2) information is available from hydrographic surveys and dredging operations. Quantities of material by-passed by natural action and quantities of material which settled down in the inlet and its adjoining entrance areas are listed in Table 2. Most data are derived from Corps of Engineers, Jacksonville District, reports. These data should, needless to say, still be considered as approximative.

TABLE 2

Predominant Drift A and By-Passed Drift B at Florida Atlantic Inlets, (cub. yds/yr. U. S. Army Corps of Engineers, Jacksonville District)

Inlet or Entrance	Predominant Drift A (cub.yds/year)	By-Passed B (cub.yds/year)
St. Mary's River	500,000	unknown
St. John's River	500,000	unknown
St. Augustine Inlet	500,000	unknown
Matanzas Inlet	500,000	almost all
Ponce De Leon Inlet	500,000	350,000
Canaveral Harbor (no tidal flow)	350,000	very little
Sebastian Inlet	300,000	200,000
Ft. Pierce Inlet	250,000	150,000
St. Lucie Inlet	200,000-250,000	30,000
Jupiter Inlet	200,000-250,000	150,000
Palm Beach Inlet	200,000-225,000	very little
South Lake Worth Inlet	150,000-200,000	40,000
Hillsboro Inlet	100,000	perhaps 30,000
Everglades Inlet	50,000	very little
Bakers Haulover Inlet	50,000	very little
Government Cut, Miami Beach	20,000	very little

South of Cape Kennedy, which is "the big robber" of material for the lower East Coast, the number of inlets increase. The quantity of drift decreases from approximately 250,000 cub.yds/yr. predominant south at the Fort Pierce Inlet to perhaps 10,000 to 20,000 cub.yds/yr. at Government Cut (Miami Beach).

It may be said that the number of inlets (and rivers) on the upper East Coast of Florida with its heavier drift is one per 40 miles, while the number of inlets on the lower East Coast with less drift is one per 20 miles (including some inlets which were cut by man replacing earlier breakthroughs or inlets cut by nature).

On the lower Gulf Coast the predominant littoral drift is very limited (about 50,000 cub.yds/yr.) thanks to low wave energy input. In four lower Gulf Coast counties (about 150 miles of shore) there is at present one inlet per 10 to 15 miles.

The upper West Coast of Florida has a few inlets only. The predominant drift is perhaps of the order 150,000 cub.yds/yr., and the tidal prisms vary strongly, thanks to the diurnal tide. It may even be non-existing for a week or two. Such situation leaves the opportunity for a few larger inlets to stay open while all smaller breakthroughs must close.

In Florida, as elsewhere, numerous inlets opened up as a result of breakthroughs caused by hurricanes or major storms. A recent example on the East Coast is the inlet which the March 9 - 11 storm in 1962 cut through the northern part of Jupiter Island (20 miles north of Palm Beach, Florida). It was not the first time that an inlet broke through in this area but they all closed. The 1962 inlet continued expanding thanks to a rather large tidal prism. Because of the fact that the inlet at the same time robbed the adjoining seashore for an increasing amount of beach sand, it was decided to close the inlet by a hydraulic dredge. The lower Gulf Coast barriers in Florida have as mentioned earlier many breakthroughs and there is hardly a place on the barriers which has not experienced a breakthrough. Some inlets received tidal prism enough to stay open. Longboat Pass (Figure 7) located north of Sarasota is such example. According to Table 1 its r-factor is 70. It was recently (1958) improved by a jetty on the north side and will undoubtedly survive for a considerable period of time in the future. It has some bay and sea shoals but tidal flow is rather strong. Mean tidal range is about 2 ft. The inlet by-passes little material and beaches on both sides have suffered. This is particularly true for the south shore on Longboat Key.

The situation is different at Big Pass located further north on the barriers at Clearwater. The r-value according to Table 1 is 139 which means that the Pass is unstable and must by-pass most material on a bar or shoal. Shoaling has accelerated in recent years, partly because of bay developments causing loss of tidal prism and Big Pass is virtually closing.

A few inlets in Florida should be offered separate attention for technical and historic reasons.

An example of a very tough inlet which, regardless of the fact that it is a typical bar by-passer, has been able to stay open for centuries is the Matanzas Inlet (Reference 4) approximately 15 miles south of St. Augustine on the Florida Atlantic Shore (Figure 8). The

Figure 7. Longboat Pass, Lower Gulf Coast, Florida

Figure 8. The Matanzas Inlet, Florida - Atlantic

inlet has a very substantial ocean shoal at its entrance with 2 to 6 ft. depth on the sea shoals. There is, however, always a channel penetrating through the shoals. The depth in the channel may vary from 4 to 8 ft., but it is deep enough to allow a vessel of the type used by the Spanish Navy of the 16th and 17th century to pass through to the lagoon at high tide. This channel is usually a single channel which because of the southward drift moves slowly from north to south and then, when an extreme southern position has prevailed for some time and a severe storm usually from the N.E. occurs, will experience a new breakthrough in the northern part of the ocean shoal. The new channel will then take over the flow and the southern channel will close. By this process a large quantity of material will be transferred at one time substituting for several years of accumulation on the updrift side of the channel which gradually forced the channel downdrift.

Ponce de Leon Inlet, Figure 9, is located in Volusia County on the east coast of Florida, about 65 miles south of St. Augustine and 57 miles north of Canaveral Harbor. Mean ocean tidal range is about 4.1 ft. at the Coast Guard Station; inside the inlet is 2.3 ft; mean range is 2.7 ft. The inlet is a natural waterway connecting the Atlantic Ocean with the Halifax River and the Indian River North. According to historical accounts the inlet has been used for navigation for more than 200 years. It is another example of a grand scale natural bar by-passer. A fan-shaped sand bar lies across the ocean entrance. The main channel across the bar changes frequently in depth, width, position, and alignment. In September 1962 the inlet channel extended in an easterly direction with depths ranging from less than 6 ft. across the bar to 35 ft. in the gorge between the land points. In April 1950 the channel extended due east with a controlling depth of 4.5 ft. In May 1949, the main channel extended northeasterly with a controlling depth of about 4 ft.

The littoral drift in the vicinity of Ponce de Leon Inlet is predominantly southerly; net southward movement is estimated to be about 500,000 cub.yds annually. Gross annual drift rates are estimated to be about 600,000 cubic years southerly, 100,000 cub.yds. northerly. Available records from 1936 to 1962 show there has been erosion north of the inlet and both erosion and accretion south of the inlet. Much of the erosion is concentrated near the inlet where the shoreline on both sides has receded. Estimated littoral drift distribution for existing conditions is summarized in Table 3. The distribution is based on the estimated net southerly drift rate. The maximum discharge Q_{max} is about 1,450 cub.yds/sec. which as indicated in Table 1 gives an r value of 345. This is indicative of the fact that the large half-moonshaped bar in the ocean in front of the Ponce de Leon Inlet transfer the greater part of the longshore drift.

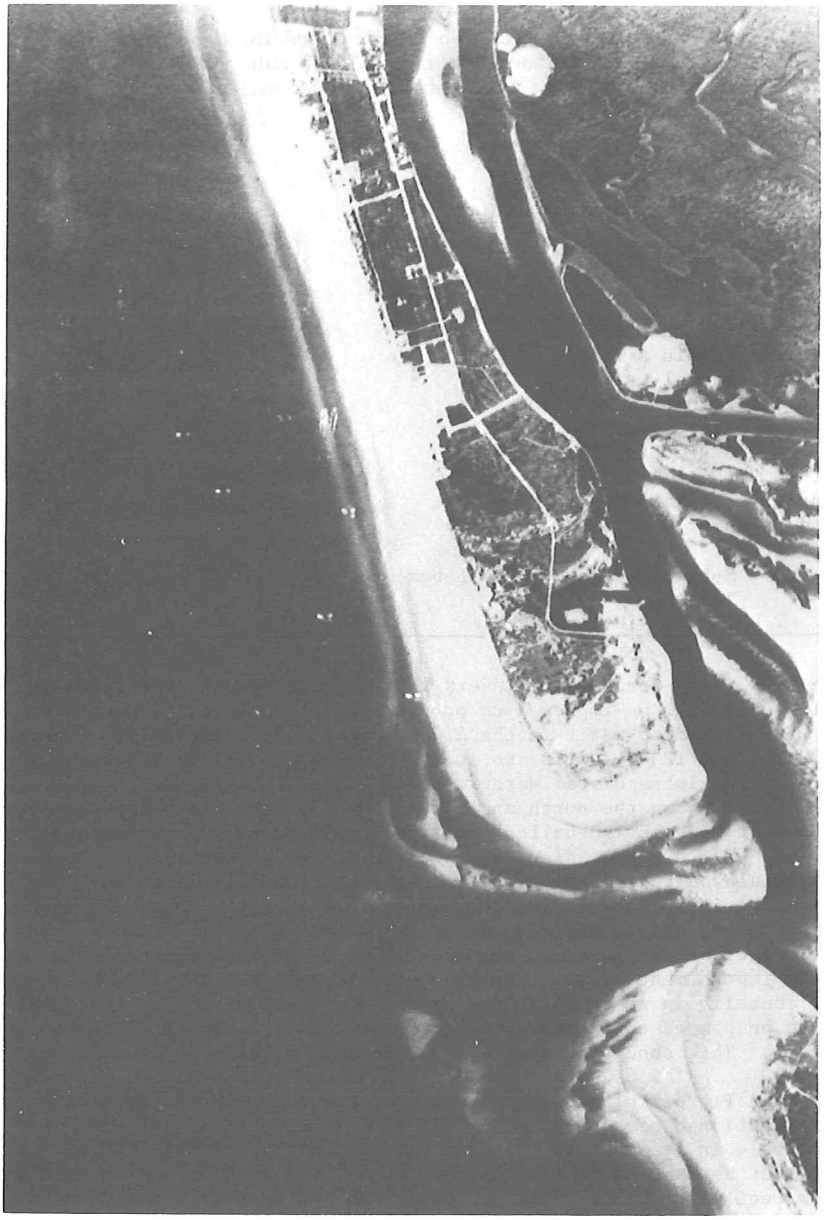

Figure 9. The Ponce De Leon Inlet, Florida - Atlantic

TABLE 3
Littoral Drift Distribution at the Ponce De Leon Inlet
Average Annual Volume - 1,000 cub. yds.
(U. S. Army Corps of Engineers, Jacksonville District)

Distributions	Existing Conditions
Net southward littoral drift to inlet	500
Transfer to south beach	
By natural processes	350
By pumping	
From initial channel dredging	0
From initial basin dredging	0
From inlet channel maintenance	0
From interior channel maintenance	0
From basin maintenance	0
Subtotal to south beach	350
Not transferred to south beach	
Retained in inlet area	50
Lost offshore	100
Subtotal, lost to south beach	150
Total	500

The U. S. Army Corps of Engineers in 1963 recommended that Ponce De Leon Inlet, Florida, be improved to provide: A channel 15 ft. deep and 200 ft. wide from deep water in the Atlantic Ocean into the inlet, thence 12 by 200 ft. and 12 by 100 ft. to Indian River North; 12 by 100 ft. southward to the Intracoastal Waterway; ocean jetties 4,200 ft. long and 2,700 ft. long on the north and south sides of the inlet respectively. The north jetty for the so-called "weir project" includes a weir section 200 ft. long with crest at elevation zero at M.L.W. Southward-drifting beach material would pass naturally over the weir and settle in an impoundment basin inside the inlet (like the Hillsborough Inlet, Broward County, southeast coast, Florida, which is mentioned later). A conventional pipeline dredge would excavate 500,000 cub.yds. initially to create the impoundment basin. The dredge would remove about 310,000 cub.yds. annually in re-dredging the basin to a depth of 18 ft. Material would then be pumped to the south beach about 2,000 ft. south of the south jetty. This should prevent any large scale erosion.

The Fort Pierce Inlet on the lower east coast of Florida (Fig. 10) should be mentioned here because in this case tidal currents play an important role in its natural transfer arrangement. Meanwhile, it is unlikely that sand would be transferred without the existence of a rather wide rock reef with 10 ft. to 12 ft. depths on the downdrift side of the inlet see (Fig. 10).

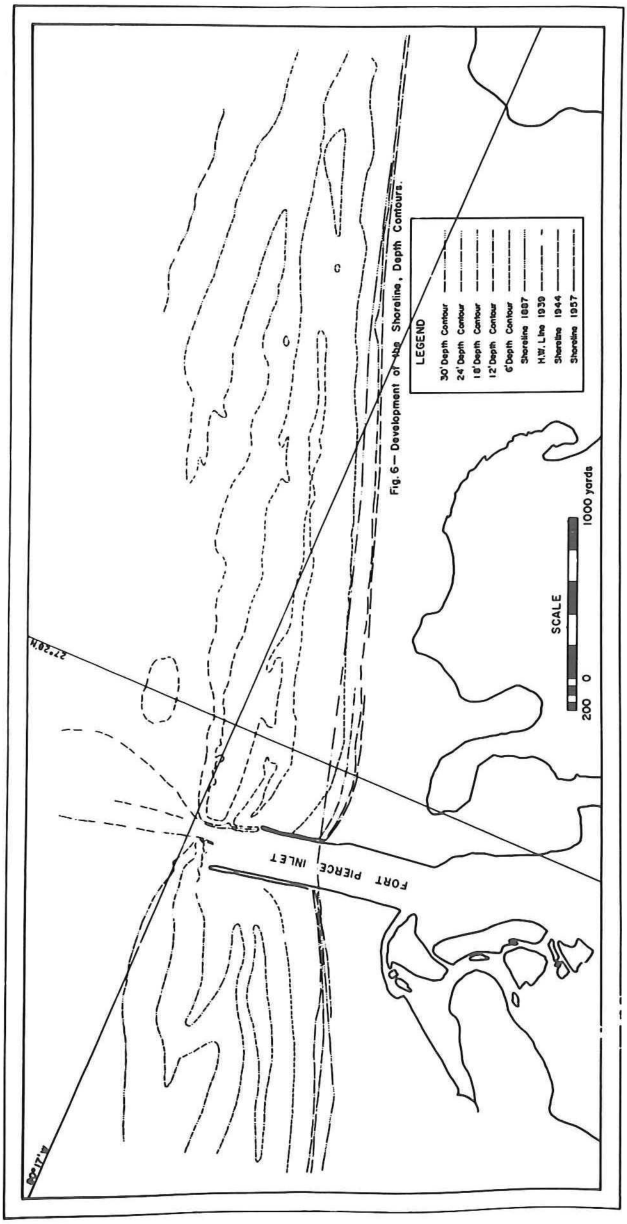

Figure 10. The Fort Pierce Inlet, Florida

Investigation conducted by the Coastal Engineering Laboratory of the University of Florida(1) are mentioned below because of its attempt to establish quantitative balance equations. The situation at the Inlet is depicted in Fig. 11. Notations refer to quantity per year as follows:

N = Net amount of littoral drift material entering the area from the north.

S = Net amount of littoral drift material leaving the area for southward drift.

s = Amount of littoral drift material deposited permanently in area north of northern jetty.

b = Amount of littoral drift material passing through and over the northern jetty into the inlet.

c = Amount of littoral drift material sucked into the inlet by flood currents.

d = Total amount of littoral drift material deposited in the inlet channel and on the bay shoals ($d = d_1 + d_2 + d_3$).

e = Amount of littoral drift material jetted out into the sea by the inlet ebb-currents.

f = Amount of littoral drift material brought out by inlet ebb-currents and deposited in deep water outside the littoral zone.

g = Net amount of littoral drift material in the offshore area south of the inlet which by-passed the inlet.

h = Amount of littoral drift material passing through or over the southern jetty into the inlet.

k = Net amount of material eroded from the beach and near offshore area south of the inlet from the southern jetty to the point where normal littoral drift has been re-established and lee-side erosion is not evident.

The following "material-balance equations" can be written:

$N = S$ (under the assumptions: similar profiles, material and wave action on both sides of the inlet)

$N = S = g + k$

$N = a + d + f + g$

$b + c + h = d + e$

This gives:

$e = b + c + h - d$

$f = k - a - d$

$e - f = b + c + h + a - k$ (the amount of material which is delivered back to the littoral coastal zone by the ebb-current through the inlet).

The different quantities presented in these equations are not well known but based on earlier surveys including information by the U. S. Army Corps of Engineers on dredging quantities at the Fort Pierce Inlet, the following data are considered fairly reliable:

$N = S =$ 200,000 to 250,000 cub.yds. (= net southward drift)

$k =$ 100,000 cub.yds.

$a =$ 20,000 cub.yds.

$d =$ 40,000 cub.yds. (26,000 cub.yds. dredged per year and the rest estimated to be deposited in the bay shoals)

$g =$ 100,000 to 150,000 cub.yds.

With $N =$ 200,000 cub.yds. and $a =$ 20,000 cub.yds. one has:

$f = 100{,}000 - 20{,}000 - 40{,}000 = 40{,}000$ cub.yds.

$e = (b + c + h) - 40{,}000$

This means that 40,000 cub.yds. a year are lost in deep water; and that because of the fact that "e" must have a positive value greater than "f" (f is only a part of e), the quantity $(b + c + h)$ entering the inlet through the jetties and through the entrance is at least 80,000 cub.yds/yr., and probably more, because the amount of material passing the extreme end of the north jetty located at about 18 ft. depth must be limited, and the bulk of the net 100,000 cub.yds. of material (g) which apparently is delivered back to the littoral zone south of the inlet must be transferred mainly by inlet ebb-currents. With a quantity of 40,000 cub.yds. to be deposited in the inlet and on bay shoals, this means that the amount of sand passing through and over the north jetty must be very high, probably about 140,000 to 180,000 cub.yds. per year.

The distribution of littoral drift along the beach and offshore bottom profile north of the inlet is not known but the offshore profiles are gently sloping. It is therefore assumed that at least 80 per cent of the littoral drift or 160,000 cub.yds. (with 200,000 cub.yds. total drift) migrate within the 18 ft. depth contour which is roughly located at the extreme end of the north jetty. With only 20,000 cub.yds per year deposited north of the inlet, 140,000 cub.yds. pass through or over the north jetty. The amount of material passing the south jetty must be much smaller than the amount passing the north jetty because of less wave action and lower storm tides from the southeast quadrant. The south jetty seems also to leak less material than the northern one. Assuming that 20,000 cub.yds. pass through the south jetty, the total amount of material passing through the jetties into the inlet is 160,000 cub.yds.

Out of these 160,000 cub.yds. plus the quantity "c" which was sucked in through the entrance, 40,000 cub.yds. is deposited in the inlet and on the bay shoals while the balance of the material amounting to 120,000 + c = e cub.yds. is delivered back to the ocean. If "c" (the total amount of littoral drift outside the 18 ft. depth contour) equals 40,000 cub.yds. a total of 160,000 cub.yds. is delivered back to the ocean where 40,000 cub.yds. are lost to deep water, while 100,000 cub.yds. drift southward and 20,000 cub.yds. ("h") is carried back into the inlet through the south jetty.

Under the assumption of a net southward littoral drift of 250,000 cub.yds. a new set of values is obtained as given in Table 4.

TABLE 4
Littoral Drift Quantities at Fort Pierce Inlet

Notation	Quantity (cub.yds.)	Quantity (cub.yds.)
N	200,000	250,000
a	20,000	20,000
(b + h)	160,000	200,000
d	40,000	40,000
e	120,000 + c	160,000 + c
c	40,000	50,000
f	40,000	40,000
g	100,000	150,000
h	20,000	20,000
k	100,000	100,000

Table 4 clearly indicates the importance of the transport of material through the jetties and the loss of material to deep water by ebb jets. The peculiar shape of the offshore bottom profile with an almost horizontal platform at 10 to 12 ft. depth is probably responsible for the fact that the inlet, to a considerable extent, works as a "natural sand transfer plant." On the downdrift side of the inlet entrance, material apparently is pushed ashore by wave action on the rock reef platform, which is closer to the shoreline farther south. If the bottom profiles had been steeper on the downdrift side more material would have been lost to deep sea and large shoals may have developed southeast of the inlet. Such shoals were not revealed by this or other surveys. The littoral drift material delivered back to the offshore littoral zone on the platform stabilizes same and decreases destructive wave action, thereby offering some protection to the beach area. In regard to this "transferring-material action," it must be considered most fortunate that the

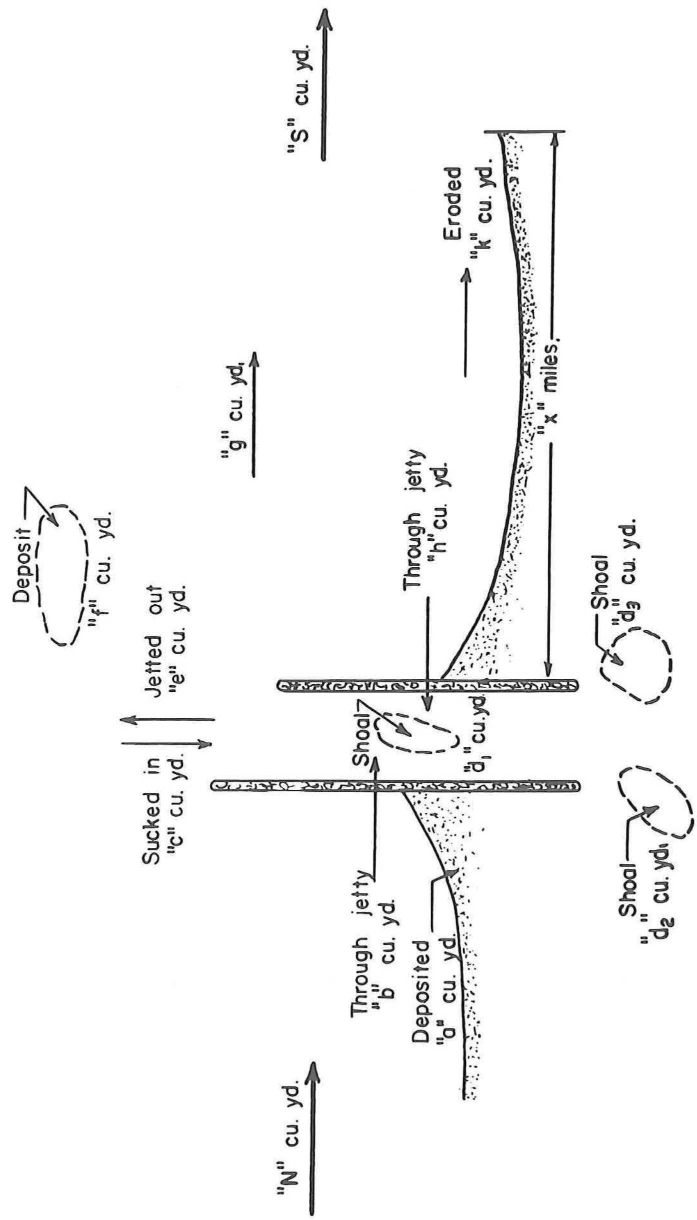

Figure 11. Material Balance at the Fort Pierce Inlet, Florida - Atlantic

south jetty is rather short with the extreme end located in only 10 ft. of water. With somewhat greater depth such action would not be very likely. A curved jetty entrance would have a similar effect, but is not always practical.

The erosion south of the inlet is still severe. A federal project has recommended a large scale artificial nourishment program with material from the bay bottom.

BY-PASSING AT HARBORS ON OPEN SHORES

Generally it may be said that by-passing at harbors located on open shores will hardly ever take place in a way which is agreeable with the requirements by navigation.(3) and (4)

An example may be found on the North Sea (Skagerrak) coast of Denmark at Hirtshals Harbor, Fig. 12 (1947). A heavy littoral drift, perhaps 500,000 cub.yds. to 1,000,000 cub.yds/yr., comes from the west by a strong wave action. Part of the drifting sand is deposited in "tongues" along the updrift jetty, while a great part passes the extended updrift jetty and deposits, by a large clockwise eddy current, in a large shoal on the downdrift side. This shoal is gradually growing larger by deposits ranging between 50,000 cub.yds. and 200,000 cub.yds/yr. Maintenance dredging is necessary in the 7m to 8 m (25 ft.) deep entrance channel to the harbor. The development in recent years shows decreasing depths on the downdrift shoal (10 ft. to 13 ft.). At the same time the shoal has extended farther downdrift with the result that the lee side shore is now being nourished from the shoal due to swell action, which apparently brings the coarsest sand material back to the shore. Accumulations however continue on the updrift side where depth contours move seaward and the port will probably experience an increasing maintenance dredging.

Another interesting example of by-passing sand by natural action at a harbor is found at the harbor of La Guaira in Venezuela. This harbor has the head of its nail-shaped updrift jetty located at 18m (60 ft.) depth. There is considerable littoral drift from the east to the west caused by heavy wave action (waves up to 20 ft. from northeast). Some years ago a tanker ran aground midway out on the updrift jetty at 30 ft. to 40 ft. depth and accumulated, in a short time, a great amount of sand behind it, demonstrating the existence of a heavy drift. Meanwhile, there has been no accumulation at the end of the jetty and it is believed that the great depth and offshore bottom steepness may be responsible for this. Reference is, in this respect, made to Cornaglia's theory (Italy, about 1900). Based on the experience with erosion of steep shores and gently sloping shores, Cornaglia claimed that a neutral line or depth exists for any condition of wave action. Outside the neutral line, drift moves seaward, inside it moves shoreward. Some laboratory experiments (e.g. at the MIT, Eagleson, 1961) have indicated certain agreements with Cornaglia's theories, which are in fact also in agreement with field experience from relatively steep shores at deep water coasts.

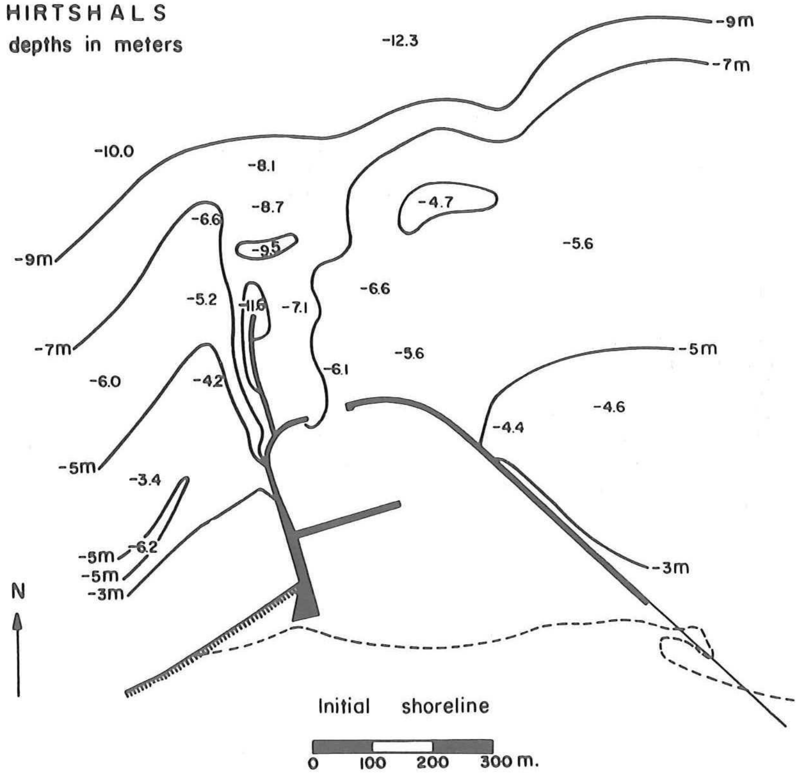

Figure 12. Port of Hirtshals, Danish Skager Rack Coast

The normal case however is that material which tries to by-pass a jettied harbor entrance may come to rest in the (dredged) entrance channel (Port of Palm Beach, Florida, Port Everglades, Florida and Government Cut, Miami, Florida), or just inside a single straight or curved jetty on a "standard shoal" (Santa Barbara, California).

Numerous model experiments have, however, been carried out through the years attempting to give harbor entrances, whether flushed by tidal currents or not, such geometrical slope that at least part of the material may by-pass the harbor by nature's own forces. Bruun and Gerritsen(4) mention several examples of that nature. One of the most recent is the improvement of the Ymuiden entrance in Holland (Amsterdam ship canal).

The Zeebrugge Harbor in Belgium (Fig. 13) should be mentioned in this connection. This harbor, protected by a 4,500 ft. long nail-shaped jetty was, for a long time, greatly bothered by silt deposits amounting to approximately 5,000,000 cub.yd/yr. The tidal range is approximately 12 ft. and the tidal currents outside the harbor up to 5 fps to 6 fps. For some time the harbor was equipped with a 1,300 ft. opening (claire-voie) permitting tidal currents to flow through the harbor basin. This was unsatisfactory. Heavy deposits, mainly silt, continued and dredging of the deposits endangered the economy of the harbor.

In order to improve this situation model experiments were conducted after World War II in Belgium (Waterbouwkundig Laboratorium) and in Holland (Waterloopkundig Laboratorium). Fig. 13 shows the general current pattern with a strong flood current. The result of the construction of the big circular jetty on the shore-side was the elimination of a large silt-depositing eddy current in the harbor basin. The amount of silt deposits was reduced to less than 50%. The remainder of the material (mainly silt) by-passes the harbor with the tidal currents.

SAND BY-PASSING PLANTS IN FLORIDA

Florida has only two by-passing plants; namely, the plants at the South Lake Worth Inlet and at the Lake Worth (Palm Beach) Inlet.

SOUTH LAKE WORTH INLET (Fig. 14)

The fixed dredging installation at the South Lake Worth Inlet is located on the seaward end of the North jetty, or about 250 ft. eastward of the M.S.L. shore line north of the inlet. It was installed in 1929, as a means of intercepting the southward littoral drift and by-passing the material across the inlet depositing it on the shore line south of the South jetty. The operation was primarily intended to supply sufficient material to nourish the heavily eroding shore south of the inlet and secondarily, to reduce shoaling at both ends of the navigation channel where the flood and ebb tide velocity was reduced to a point where deposition took place. The inlet, itself, was constructed in 1927 to provide exchange of bay waters and sea water for the south end of the Lake Worth and to give access to the ocean for fishing boats and pleasure crafts. The channel is approximately 125 ft. wide, and 600 ft. long. It accommodates crafts drawing up to 6 or 8 ft. The top

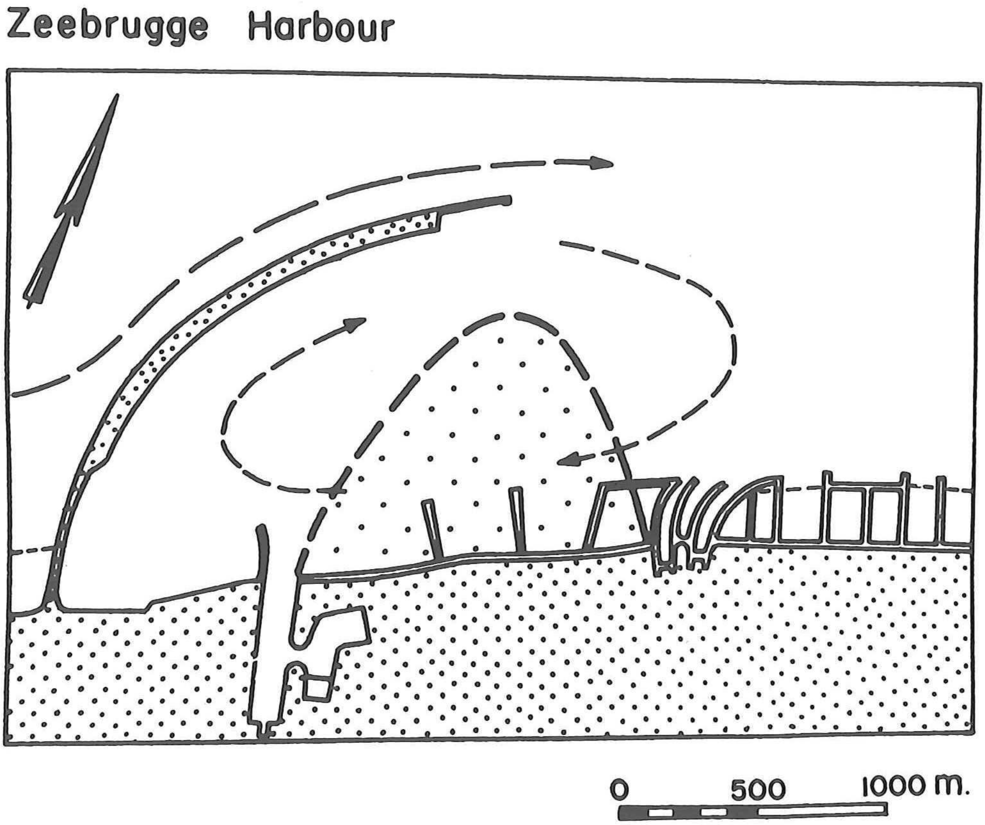

Figure 13. Zeebrugge Harbor, Belgium

Figure 14. The South Lake Worth Inlet, Florida

elevation of the steel sheet piling and concrete jetties is about 12 ft. above mean low water.

The sand transfer facility was initially constructed in 1929. In 1937, the plant was reconstructed, and at this time it was provided with an 8 inch suction, 6 inch discharge, and 65 horsepower diesel-driven centrifugal pump, and approximately 1,200 ft. of 6-inch discharge line. The discharge line is carried across the inlet on a concrete bridge (highway A1A). The pump was operated continuously on an as-needed basis from 1937 until 1942, at which time the pumping was ceased due to a fuel shortage during World War II. At the end of World War II (1945), pumping was resumed, and in 1948, the pump was reconstructed to a larger size and provided with a 10 inch suction, 8 inch discharge, and an approximately 300 horsepower diesel engine, and 700 to 750 ft. of mechanical joint cast-iron discharge line. The power plant has been replaced once since 1938 (1955) and pump parts have been replaced on a preventative maintenance basis. The discharge pipeline on this plant is rotated approximately on a two-year interval and is generally replaced entirely after three rotations, or six years. The plant has proved to be a dependable asset in transfer operations, notwithstanding its "ugly duckling" look.

TABLE 5
South Lake Worth Inlet
By-Passed Quantities, 1960-1965

Period	Cub.Yds. During Period	Accumulative Cub. Yds.
Oct. 1960 thru Sept. 1961	31,737	31,737
Oct. 1961 thru Sept. 1962	45,339	77,076
Oct. 1962 thru Sept. 1963	88,366	165,442
Oct. 1963 thru Sept. 1964	70,300	235,742
Oct. 1964 thru March 1965	20,520	256,262

The operational costs during the 1960 to 1965 period has been about $25,000 per year. The production record during this period is set forth

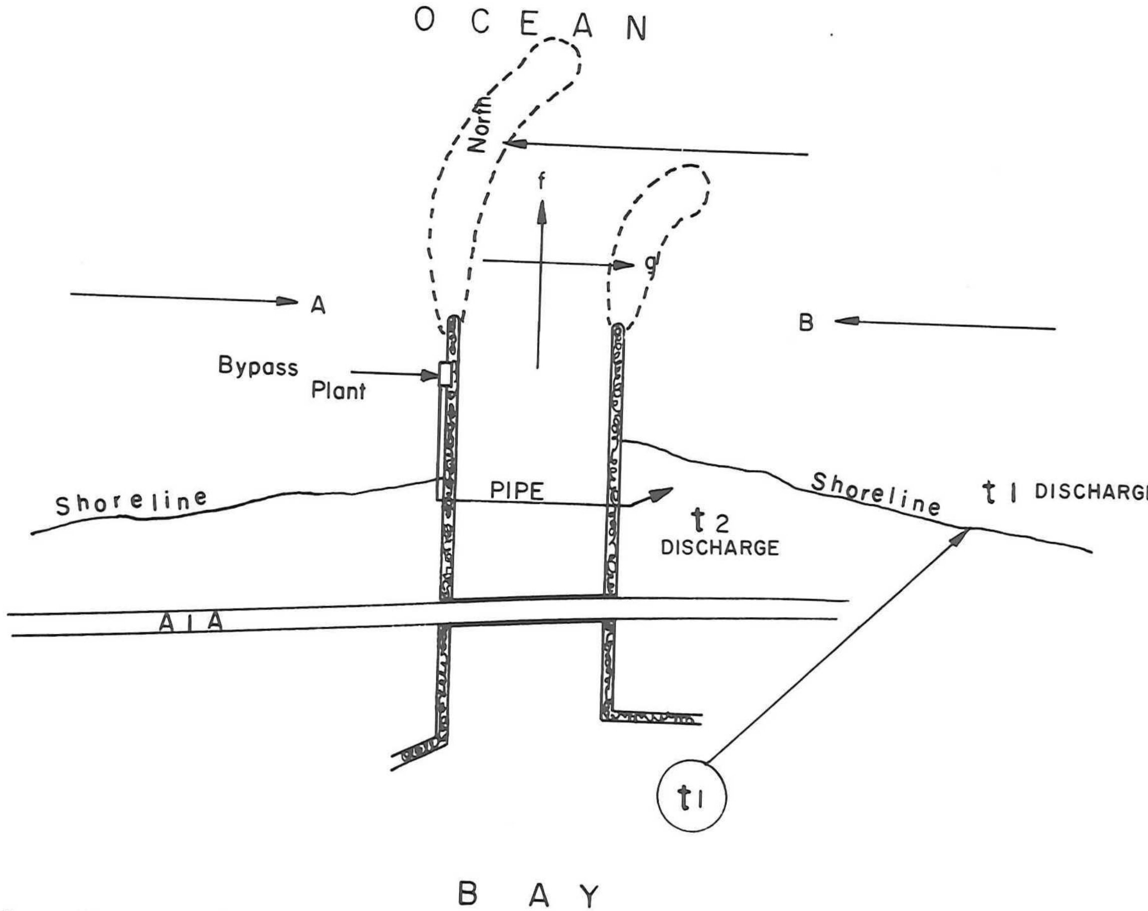

Figure 15. Material Balance at the South Lake Worth Inlet, Florida. Extension of Jetties Indicated by Dotted Lines

on Table 5. An estimate of reconstruction of this plant to today's size and at today's construction costs, approximates at least $100,000.00. The operational unit cost for the current expenditures for the selected period (1960-1965) is $.41 per cub.yd. If the plant is amortized over a 40-year period, using current construction costs, five cents must be added to the current operational figures bringing the approximate total unit cost to $.46 per cub.yd. This amount compares very favorably to the contract cost of doing dredging work in approximate 100,000 cub.yds. quantities in tidal inlet waters. The operation must be said to have been successful. Shore line recessions for a 10,000 ft. section south and north of the inlet for the period 1929 to 1955 are indicated in Tables 6 and 7. The average recession was 58 ft. south of the inlet and 33 ft. north of the inlet. These values (2-3 ft/yr) correspond closely to the shore line recession calculated by Bruun as a result of the sea level rise of about 1/4 inch per year during the period 1930-1950, (5) and (8). Partly because a recommended enlargement of the plant recently gave rise to considerable controversy between the muncipalities on both sides of the inlet and partly because of its age and the pioneer work done on the development and proper function of it, the material balance at the inlet including the by-passing, which takes place partly by the pumping plant and partly by hydraulic dredge from the bay shoals is mentioned in detail below.

Movements of shore line and depth contours on both sides of the inlet - Table 6 shows shore line movements in various sections totaling 10,000 ft. of the shore north of the inlet for the 1929-1955 period. A similar comprehensive survey has not been made since 1955.

It may be seen that shore line has moved seaward for a short distance (1,250 ft.) north of the inlet. Next follows a neutral area and then recession. The 6 ft. contour shows the same pattern enlarged. Accumulation on the updrift side of a littoral barrier will almost always cause a local shore line recession on the updrift side beyond a distance of 4 to 6 times the barrier length.

Table 7 shows similar figures for a 10,000 ft. section south of the inlet. Just south of the inlet the material by-passed has caused seaward movement of the shore line, but shore line recession has taken place south of here for the major part of the 10,000 ft. section. The recession has the same order of magnitude as north of the inlet. The movement of the 6 ft. contour shows the formation of a plateau (shoal) south of the inlet (about 1,200 ft.). For the remaining part, the 6 ft. contour has receded in some exaggerated scale compared to the shore line. This is a quite normal development.

The figures of Tables 6 and 7 reveal that the direct influence of the inlet has mainly been local. As already mentioned above, the average shore line recession north of the inlet was only 33 ft; south of the inlet 58 ft. in 1929-1955. The situation may have changed in the disfavor of the north beaches in recent years, but erosion has accelerated on all shores on the East Coast since 1960, undoubtedly as a result of the rise of sea level before 1960 (5).

TABLE 6
Shore Line Movements North of the Inlet, 1929-1955
(Corps of Engineers, 1956)

Distance from North Jetty ft.	0 - 1,250	1,250 - 2,500	2,500 - 5,000	5,000 - 10,000
Changes ft.	+62	0	-41	-62

Note: - means moving shoreward

\+ means moving seaward

TABLE 7
Shore Line Movements South of the Inlet, 1929-1955
(Corps of Engineers, 1956)

Distance from South Jetty ft.	0 - 1,300	1,300 - 10,000
Changes ft.	+105	-83

An indirect influence of the inlet is the lee-side erosion which occurs when northward drift for a certain period of time prevails and material is transferred across the inlet in quantities which may be in excess of what it should be during the abnormal conditions.

<u>Material balance at the inlet</u> - Outside the 1,000 ft. shore on both sides of the inlet.

With reference to Fig. 15, the equilibrium condition for the sand budget at the South Lake Worth Inlet is established using the following terminologies:

A = southward drift in cub.yds/year

B = northward drift in cub.yds/year

$\frac{A}{B} = \alpha$ = the ratio between southward and northward drift.

g = balance of material which is by-passing the inlet by natural action south and north in cub.yds/year

t_1 = material by-passed by dredging from bay shoals (cub. yds/year)

t_2 = material by-passed by pumping from the north jetty (cub. yds/year)

f = material lost to deep water because of flushing by ebb currents (cub.yds/year)

Equilibrium equation for the North Shore is

$$(A-B) - g - t_1 - t_2 \geq 0$$

Equilibrium equation for the South Shore is

$$-(A-B) + g + t_1 + t_2 - f \geq 0$$

Inasmuch as A and B vary and this may have some influence on g and f, it is not possible to fulfill these equations all the time, but they should not deviate too much from 0 any time. It will always be a deficit thanks to f (loss of material to deep water).

In report on model study for the South Lake Worth Inlet by the Coastal Engineering Department of the University of Florida, the following figures are used based on experience, including data published in the cooperative beach erosion study report by the U. S. Army Corps of Engineers and the Palm Beach County (1947 and 1956) and data on dredging by the Inlet District: Predominant drift, order of magnitude - approximately 200,000 cub.yds/year (may drop to 150,000 cub.yds/year) transfer of material from bay shoals 40,000 cub.yds/year (varying 30,000 cub.yds/year to 90,000 cub.yds/year) and transfer by the present by-passing plant 70,000 cub.yds/year (varying ± 10,000 cub.yds/year). See Table 5.

Different possibilities are now considered. Table 8 gives the quantities of material moving south (A) and north (B) under the assumption that A-B = 240,000 cub.yds/year, 210,000 cub.yds/year, 180,000 cub.yds/year, and 150,000 cub.yds/year with $\alpha = \frac{A}{B}$ varying from 2 to 4.

TABLE 8
Assumed Littoral Drift Quantities Moving North and South of Palm Beach, Florida
(cub. yds per year)

$$\alpha = \text{Ratio of Drift} \frac{\text{Southward}}{\text{Northward}} = \frac{A}{B}$$

	A-B	A	B	Is it realistic ?
	240,000 210,000 180,000 150,000			
$\alpha = \frac{2}{1}$	240,000	480,000	240,000	No
	210,000	420,000	210,000	No
	180,000	360,000	180,000	No
	150,000	300,000	150,000	?
$\alpha = \frac{3}{1}$	240,000	360,000	120,000	No
	210,000	315,000	105,000	No
	180,000	270,000	90,000	Yes
	150,000	225,000	75,000	Yes
$\alpha = \frac{4}{1}$	240,000	320,000	80,000	No
	210,000	280,000	70,000	Yes
	180,000	240,000	60,000	Yes
	150,000	200,000	50,000	Yes

Only five of the combinations listed in Table 8 seem to be realistic, including one A-B = 210,000 cub.yds/year (α = 4), two A-B = 180,000 cub.yds/year (α = 3 or 4) and two A - B = 150,000 cub.yds/year (α = 3 or 4).

These possibilities are considered in Table 9 referring to the present situation assuming that $t_1 + t_2$ varies from 100,000 cub.yds/year to 160,000 cub.yds/year, g = 50,000 cub.yds/year (which may be on the high side) and f(lost to deep water) varying from 30,000 cub.yds/year to 10,000 cub.yds/year as $t_1 + t_2$ increases. Comparing the equilibrium condition for the north and the south shore this table reveals that the condition with $t_1 + t_2$ = 130,000 cub.yds/year, g = 50,000 cub.yds/year and f = 20,000 cub.yds/year seems to be tolerable. If f > 20,000 cub.yds/year, the quantity by-passed south could be increased to improve the north/south balance.

Present condition is that about 70,000 cub.yds/year averagly is by-passed by the pumping plant and about 45,000 cub.yds./year is by-passed from the bay shoals (perhaps a little less or 40,000 based on data going back to 1950) total 115,000 cub.yds/year. It, therefore, does not seem to be necessary to cut down quantities by-passed by the plant below 70,000 cub.yds/year.

TABLE 9
Material Balance at the South Lake Worth Inlet
Present Conditions
Quantities in cub. yds/year

	North $(A-B) -g -t_1 -t_2$			South $-(A-B) +g +t_1 +t_2 -f$		
	NO	OK	OK(?)	NO	OK	OK(?)
A-B = quantity cub.yds/yr $\alpha = \frac{A}{B}$	$t_1 + t_2$ =100,000 g = 50,000	$t_1 + t_2$ =130,000 g = 50,000	$t_1 + t_2$ =160,000 g = 50,000	$t_1 + t_2$ =100,000 g = 50,000 f(lost) 30,000	$t_1 + t_2$ =130,000 g = 50,000 f(lost) 20,000	$t_1 + t_2$ =160,000 g = 50,000 f(lost) 10,000
A-B = 210,000	+60,000	+30,000	-0,000	-90,000	-50,000	-10,000
α = 4	no	no	yes	no	no	yes
A-B = 180,000	+30,000	-0,000	-30,000	-60,000	-20,000	+20,000
α = 3	no	yes	yes	no	yes	yes
A-B = 150,000	-0,000	-30,000	-60,000	-30,000	+10,000	+50,000
α = 3	yes	yes	no	yes	yes	no
A-B = 180,000	+30,000	-0,000	-30,000	-60,000	-20,000	+20,000
α = 4	no	yes	yes	no	yes	yes
A-B = 150,000	-0,000	-30,000	-60,000	-30,000	+10,000	+50,000
α = 4	yes	yes	no	yes	yes	no

No means: cannot be accepted
Yes: acceptable with certain precautions
OK: seems in general to be acceptable
OK(?): questionable, should probably not be accepted

TABLE 10
Material Balance at the South Lake Worth Inlet
Improved Conditions
Quantities in cub. yds/year

	North $(A-B) - g - t_1 - t_2$		South $-(A-B) + g + t_1 + t_2$	
	No	OK	No	OK
	$t_1 + t_2$ =130,000 g = 20,000	$t_1 + t_2$ =160,000 g = 20,000	$t_1 + t_2$ =130,000 g = 20,000 f(lost) 10,000	$t_1 + t_2$ =160,000 g = 20,000 f(lost) 10,000
A-B = 210,000 α = 4	+60,000	+30,000	-70,000	-40,000? pump 170,000
A-B = 180,000 α = 3	+30,000	-0,000	-40,000	-10,000
A-B = 150,000 α = 3	-0,000	-30,000	-10,000	+20,000
A-B = 180,000 α = 4	+30,000	-0,000	-40,000	-10,000
A-B = 150,000 α = 4	-0,000	-30,000	-10,000	+20,000

No means: cannot be accepted
Yes: acceptable with certain precautions
OK: seems in general to be acceptable
OK(?): questionable, should probably not be accepted

Table 10 demonstrates the condition after improvements based on report of May 1964 by the Coastal Engineering Department have been made. The $t_1 + t_2$ = 100,000 cub.yds/year case must now be disregarded because the jetties now are so long, that the inlet probably constitutes an almost complete littoral barrier to longshore drifts. It is however assumed that about 20,000 cub.yds/year still by-passes the inlet (mostly in suspension during storms). Furthermore, it is assumed that f = 10,000 cub.yds/year are lost offshore. The figures of Table 10 demonstrate that a total transfer of about 160,000 cub.yds/year seem to fulfill the equilibrium condition fairly adequate.

The situation at the inlet, however, is that the magnitude of drift (A and B) as well as the ratio between southward (A) and northward (B) drift (Table 8) varies from year to year and it was therefore considered desirable to establish certain procedures for by-passing which will allow corrections on the material balance budget, in case fluctuations in the littoral drift pattern beyond average conditions should occur.

The following procedure was suggested and accepted:

Starting at 1,000 ft. north and south of the inlet a base line is established running north and south. The location of the M.S.L. shore line and the profile up to 6 ft. depth is measured at 21 points 200 ft. apart (4,000 ft.) every year in April or May on a calm day. The next following year a similar survey is made. If shore line or profile has moved out a_N ft. (a_N sq. ft.) on the north side and receded a_S ft. (a_S sq. ft.) on the south side, material from the bay shoals of the inlet should probably be dumped on the south side. If the situation is the opposite (shore line recession on the north side, accumulation on the south side) material from the bay shoals could in the period when drift is northward be dumped on the north side where it is most needed. It is probably not practical to interrupt the operation of the by-passing plant because this could mean that more material than desirable is accumulated along the north jetty and possibilities for losses of material to deep water or to the inlet channel thereby increases. There is, needless to say, also the possibility that the shore line moves east on both sides or moves west on both sides of the inlet. In such case, material from the shoals could be distributed in inverse proportion to the movements. It may, however, be impractical to do so, unless quantities are large. If dredging takes place in the summer and spring season, drift is northward and sand dumped on north side of the inlet (but not in the corner between the jetty and the present shore line) will migrate northward replacing material eroded.

<u>Material balance at the inlet</u> - Inside the 1,000 ft. shore on both sides of the inlet.

With respect to the development of the shore line just north of the inlet, there has been some concern that the hole dredged by the pumping plant could give rise to recession of the shore line.

Probings have shown that the surface of the rock is located in about -4 ft. at the present plant and at -9ft. at the new plant. On the free open shore in similar material at Jupiter Island, the -10 ft. depth contour is located from 300 to 500 ft. from the M.S.L. shore line. At Palm Beach at groin protected shore, the -10 ft. contour is located about 300 ft. from the M.S.L. shore line. North and South of the South Lake Worth Inlet surveys demonstrate that the -9 ft. contour is located about 300 ft. from the shore line.

The shore line just north of the north jetty is normally located 100 ft. further seaward than the general shore line extending 600 to 800 ft. north of the inlet. The shore line just north of the inlet should not be allowed to recede beyond the line of the general shore line north of the inlet which matches with the general shore line south of the inlet. It should rather be located a little outside that line all the time.

Inasmuch as the present by-passing plant apparently has never been able to remove the triangular shaped deposit north of the north jetty, the same will be the case with the new transfer plant if the bottom slope is not allowed to be steeper than about 9 ft. in 200 ft. as on the present shore. This, in turn, means that the depth straight north of the extreme end of the present north jetty must never excede 9 ft., which happens to be the average elevation of the rock bottom in this area. Practical tests will demonstrate to which extent dredging may result in a local shore line recession beyond the desirable limit. It is, however, obvious that the new plant because of the fact that it is placed about 100 ft. further seaward than the present plant will hardly be able to make the new situation worse than the present if sand is always dredged to capacity in the N.E. sector, before dredging is started in the N.W. sector. If necessary, dredging in the N.W. sector could be slowed down during strong N.E. storms, which may decrease steepness of the bottom profile, beyond normal steepness. The Inlet District has decided to cut down boom length of the suction pipe from 80 ft. to 50 ft. and the high elevation of the rock bottom at the by-passing plants (-4 ft. to -9 ft.) should prohibit any extensive influence of the dredging on the adjacent bottom and beach.

The erection of a cofferdam was proposed by the Town of Manalapan. The intention of the cofferdam (sheet pile wall around the borrow pit for the pumping plant) was to hinder sand from a wide area in sloughing down in the borrow pit letting the walls, rather than the rock bottom, determine the limit of the area influenced by the trap. In making this suggestion, an important factor was, however, ignored, namely the fluctuation of the bottom profile which is very considerable up to 6 to 8 ft. or more in the rather coarse high permeability sand. The result of such fluctuations would be that the cofferdam sometimes would be sticking up above the bottom and sometimes would be buried in the bottom. In case of the former, sand will still drift towards the dam carried by the longshore currents.

Most sand transport takes place within 1 ft. above the bottom. Inasmuch as the limiting velocity for sand movement is between 1/2 and 1 ft/sec. longshore currents combined with wave action may be able to move sand where either one of them was too weak to initiate movement. Any hindrance to the current as for example, a vertical-walled cofferdam will increase current velocity and cause scour along the wall by concentration of currents. This could easily result in a continued loss of sand which would move to deeper waters and possibly be carried past the jetty or jetties and deposited in the inlet by flood currents or jetted out in deep waters by ebb currents. Among other things, the sand is neither preserved for the north nor for the south beach, but is carried away where it does no good to anybody. This situation will mainly occur at N.E. storms, where it in particular is unfortunate.

The cofferdam could be built with adjustable walls, but it would require a diver to make the adjustment and it would not be possible to adjust when it is most needed.

A groin, for example, of the adjustable type on the updrift side of the north jetty may have a similar effect as a cofferdam. It will be able to hold sand back to some extent under normal conditions, but during storms it may create transversal currents which would carry sand seaward where it would have less purpose than on the beach. Generally, it may be said that any type of structure perpendicular to shore would have an adverse effect, which is very non-desirable in this case. This will be understood from the following paragraph.

PALM BEACH INLET

The fixed dredging installation at this inlet (Fig. 16) was installed principally in the years 1957 and 1958. The design and construction of the plant was preceded by a number of studies; one by the designing firm in 1954; one by the State Board of Conservation in 1955; and a number of shorter reports by local professional engineers preceding the longer and final reports. Basically, this plant consists of a 12-inch suction, 10-inch discharge, 400 horsepower electrically driven motor and pump combination; a 17,000 gallon emergency flushing tank and approximately 1,700 ft. steel and rubber discharge line. The submarine portion of the discharge line that runs beneath the ship channel at 28 ft. below mean sea level is rubber; the remainder of the line is steel. The section of rubber line was decided upon due to impending deepening of the ship channel and to facilitate removal in time of need. The rubber hose is of a smooth bore type, constructed of pure gum rubber and multiple layers of canvas duck. It is highly resistant to internal wear and guaranteed to be resistant to invasion by marine borers. The steel section of the line is extra heavy, 0.5 inch wall thickness dredge pipe. The operating house at this installation has an operation deck and a machinery deck; the former is at 11.5 ft. above mean low water the latter is 1.0 below mean low water. The 17,000 gallons emergency flushing tank is necessary to preclude inadvertent plugging of the submarine section of the discharge line and thereby, the inheritance of an approximate $20,000 immediate maintenance problem. Electrical power at this installation was more or less dictated by local abutters who chose noise generation as one focal point of their general objections to construction of the plant. One other major objection to this installation, that of possible

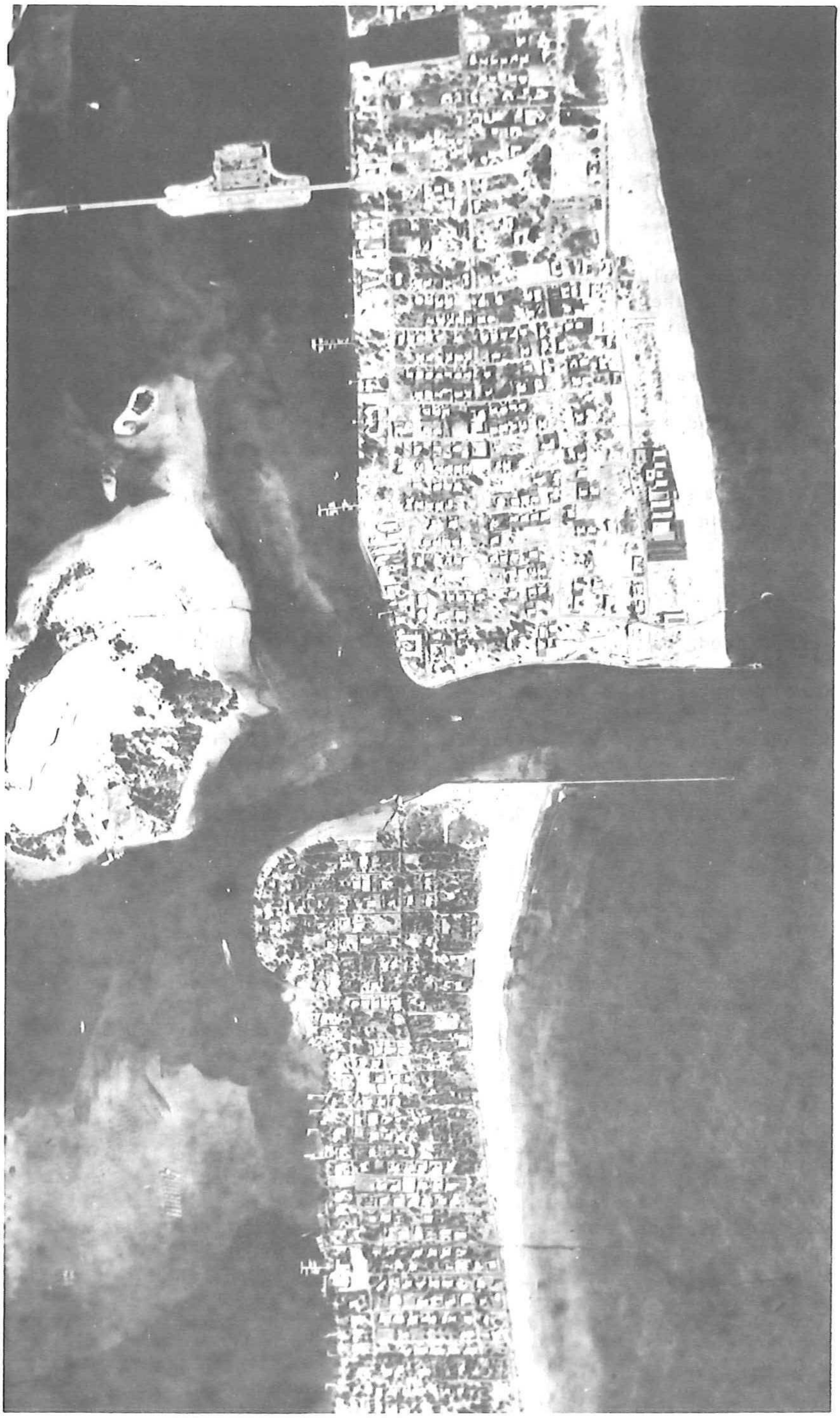

Figure 16. Lake Worth (Palm Beach) Inlet, Florida

general landward recession of the shore line due to pumping operations, resulted in the installation of a submerged sheet-pile groin just north of the plant. This device is supposed to prevent the pump from transferring any more sand than comes across the groin, but has proved to be a very undesirable feature because it deprives the plant of sand at the same time as it leads the sand to deeper water where it contributes to shoaling in front of and in the inlet channel. This pump has been operated continuously on an as-needed basis since its initial operation in September 1958, with lapses due to several failures in service and other factors. Since this was a fully planned installation, which was constructed at a time when people lived in close proximity, aesthetic values were taken into consideration. This plant therefore is more handsome than the smaller "jerry-built" installation at the South Lake Worth Inlet.

TABLE 11
Lake Worth (Palm Beach) Inlet
By-Passed Quantities 1960-1965

Period	Cub.yds. During Period *	Accumulating Cub.Yds.
Oct. 1960 thru Sept. 1961	42,730	42,730
Oct. 1961 thru Sept. 1962	48,300	91,030
Oct. 1962 thru Sept. 1963	110,601	201,631
Oct. 1963 thru Sept. 1964	70,350	271,981
Oct. 1964 thru March 1965	9,975	281,956

*Planned transfer was up to 200,000 cub.yds/yr.

The additional costs reflected in the unit price of moving material at the Palm Beach Plant, is somewhat attributable to the elaborate safeguards in the form of the emergency flushing installation and also the luxury of all electrical automatic operations. Included among observations of the operation of both of these plants, there are facets of the Palm Beach installation that would not be included in any new pump. Also the submerged ("political") groin has, as mentioned above, undoubtedly served a purpose of decreasing by-passing quantities and increasing the loss of material to deeper waters and to the inlet channel where part of it has to be picked up again by hydraulic dredges. This problem will now be subject to tracer studies by fluorescent material. A large shoal has accumulated just S.E. of the inlet entrance and may indicate where the material actually goes instead of being used for nourishment.

SAND BY-PASSING ARRANGEMENTS IN FLORIDA

At a few inlets in Florida material has been by-passed more or less regularly using a normal hydraulic dredge. The most known example of this is the Hillsboro Inlet at Pompano Beach 25 miles north of Miami. This inlet is protected by an almost shore-parallel reef on its north side. The reef is low, and sand from the north side (updrift) beaches spills over the reef, and deposits inside the jetty where a hydraulic dredge usually at intervals of about 1 to 2 years picks it up and transfers it to the south side beaches (as now proposed for Ponce De Leon Inlet, South of Daytona Beach on the Upper East Coast).

During the past eight years the inlet has been dredged with a small 8 inch dredge with a hydraulically driven cutter head. The main pump having an 8 inch suction and an 8 inch discharge pipe is powered by a Caterpillar Diesel engine. This dredge is owned by the Inlet District and while operating has a crew of 3. The sand is dumped south of the inlet from the jetty on up to 300 ft. south of it with distribution of sand further south carried on by natural forces.

The quantities which have been dredged for the past 10 years are indicated in Table 12.

TABLE 12
Quantities Dredged and Transferred at Hillsboro Inlet
Pompano Beach, Florida

1955	60,000 cub.yds.	1960	46,000 cub.yds.
1956	25,000 cub.yds.	1961	32,000 cub.yds.
1957	55,000 cub.yds.	1962	112,000 cub.yds.
1958	75,000 cub.yds.	1963	105,000 cub.yds.
1959	40,000 cub.yds.	1964	68,000 cub.yds.

The figures shown above for 1955 and 1956 were accomplished with private dredges hired for a specific job paid for with voluntary contributions. Starting in 1957 the dredging was done with the district dredge on a rather hit-or-miss basis through 1961. For that 5-year period the dredge was brought from a "mothball" condition only when the inlet was almost impassible.

Since that time dredging has been continued on a more regular schedule which basically provides for the dredging of any shoal areas as well as the settling basin which is filled over the sand spillway on the north side. In general, the schedule is developing whereby dredging will start during the latter part of April and proceed through the middle of the summer, at which time the dredge is temporarily laid up ready to start again after the fall storms have dumped sand into the settling area inside the north jetty. In 1964, however, there was no dredging after the fall storms which will probably be reflected in larger amounts in 1965.

A model study of the Hillsborough Inlet has been completed (Coastal Engineering Department, University of Florida). The south jetty has been reconstructed and the north jetty is going to be in the near future, based on the results of the experiments. The basic principle for transfer of material across the inlet will, however, be the same including some intended improvements of the weir and trap arrangements in the north jetty.

TIDAL INLETS STABILITY CONSIDERATIONS

This brief section was included because of the close relation between by-passing ability and inlet channel stability.

A = area of inlet channel cross sectional (m^2 or ft^2)

A_c = area of inlet channel cross section corresponding to τ_c (m^2 or ft^2)

A_{sm} = area of inlet channel cross section corresponding to τ_{sm} (m^2 or ft^2)

C_1, C_c = Chezy factors ($m^{1/2}$/sec) or (ft.$^{1/2}$/sec.)

d = grain size diameter (mm or in)

D = depth of channel (m or ft.)

g = acceleration of gravity (m/sec.^{2} or ft./sec.^{2})

M_{net} = predominant drift or net quantity of drift (cub. yds/yr. or m^3/yr.). M_t = total drift to the inlet.

p = percentage

q_s = rate of bed load transport (kg/m or lbs/ft) in channel

Q_{max} = peak discharge of inlet flow (m^3/sec. or cub.yds/sec.)

V = quantity of material flushed by the inlet currents in one cycle (cub.yds/cycle or m^3/cycle)

S_e = energy slope

S_s = specific weight of sediment (kg/m^3 or lbs/ft^3)

α, β, β' = special factors

ρ = density ($\frac{kg\ sec.^2}{m^4}$) or (slugs/ft.^{3})

γ = specific weight of water (kg/m^3 or lbs/ft^3)

W = width of channel (m or ft.)

τ_m = shear stress between flow and bottom (kg/m^2 or lbs/ft^2)

τ_c = critical shear stress for start of material movement (kg/m^2 or lbs/ft^2)

τ_{sm} = determining shear stress for bottom stability (kg/m^2 or lbs/ft^2)

RELATION BETWEEN FLUSHING ABILITY AND LITTORAL DRIFT

An attempt is made to describe the development of a tidal inlet as a function of the total littoral drift from both sides to the inlet (M_t), flushing ability (quantity of material) of the inlet currents (V) and bottom shear stresses in the inlet channel (τ) influencing the cross sectional area for flow. Figure 17 depicts the situation with $V \geq pM_t$ and $V \leq pM_t$ versus τ/τ_{sm}, where τ_{sm} is the determining shear stress for bottom stability. Condition for establishment of equilibrium is partly that $V = pM_t$, and partly that the inlet channel develops a bottom shear stress which is able to keep the channel free of deposit without scouring it beyond desirability.

Research by Bruun and Gerritsen(3) based on tidal hydraulics computations using Keulegan's simplified method (11) has demonstrated that bottom shear stresses under "equilibrium" or "stability conditions" may vary from 0.35 kg/m^2 or 0.07 lbs/ft^2 (light littoral drift) to 0.5 kg/m^2 or 0.10 lbs/ft^2 (heavy littoral drift) for normal beach sand (0.15 to 0.3 mm) all depending upon the magnitude of sand drift to the inlet from the sides. If the shear stress either increases considerably above the determining τ_{sm} or decreases considerably below τ_{sm} this means that the situation is unstable and is attempting to approach a more stable condition. The final result of such development may be a more stable inlet or it may also mean that the inlet either develops as a non-scouring channel, if the transfer of littoral drift material to the inlet is small or that the inlet simply starts closing because of overwhelming drift to the inlet channel from the sides.

Reference is made to Figure 17, assuming first that the inlet starts with $\frac{V}{pM_t} > 1$, which means that more material is flushed out of the inlet than deposited in it. Consequently, the cross section will enlarge (the inlet widens) and the $\frac{\tau}{\tau_{sm}}$ ratio will decrease. If M_t is small the inlet may develop towards a non-scouring channel. If M_t is larger or very large the inlet may develop a stable channel with a τ_{sm} value in accordance with the outside input of sediment load.

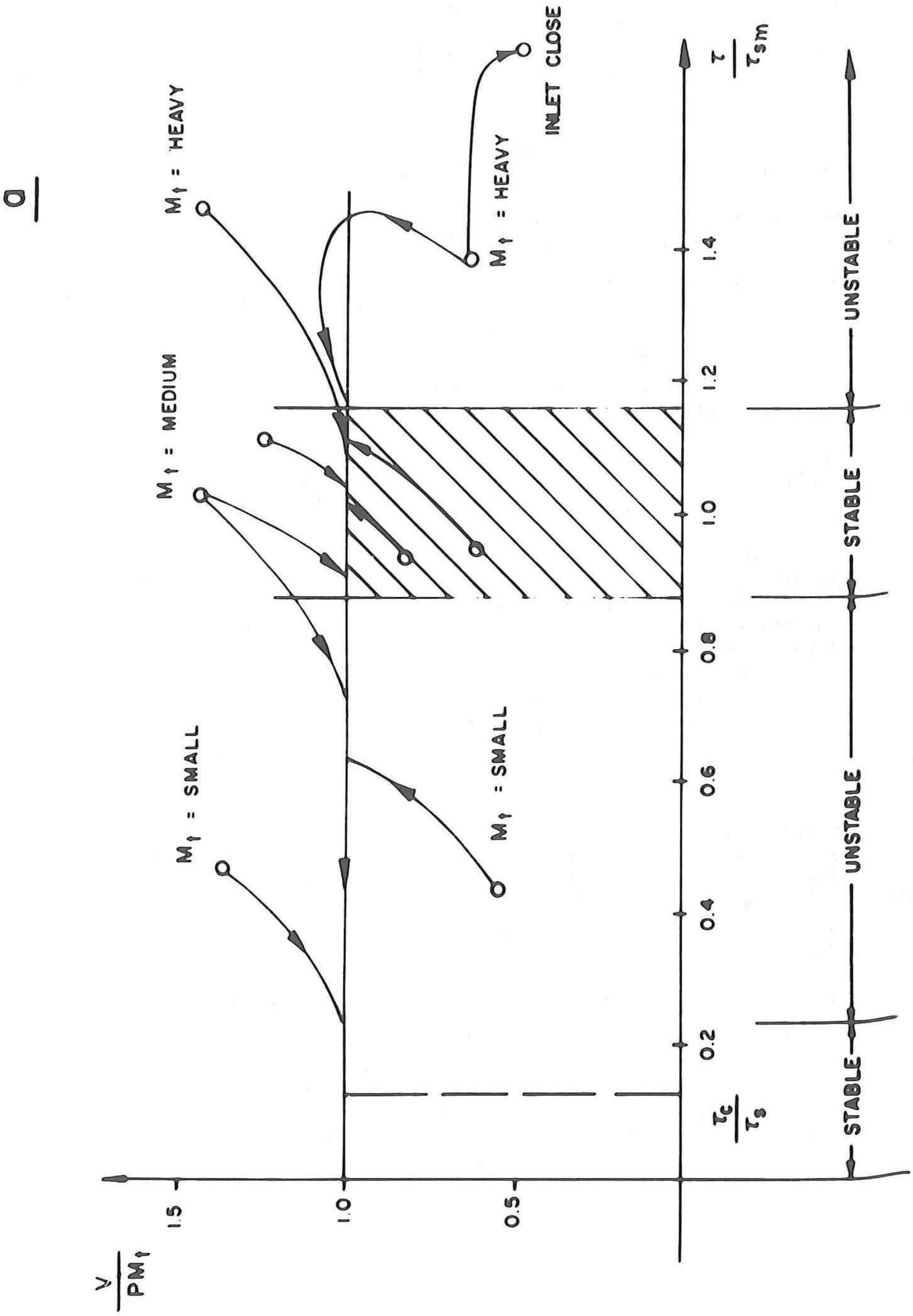

Figure 17. V/pM versus $\frac{\tau}{\tau\ \mathrm{sm}}$ for Inlet Channel

In case $\frac{V}{pM_t} < 1$, $\frac{\tau}{\tau_{sm}}$ must increase because the littoral drift encroaches upon the inlet channel. The cross section decreases (the inlet narrows). If M_t is relatively small a stable situation may not be reached but the channel may develop slowly towards a non-scouring condition. If M_t is very large the channel may or may not develop towards stability. If the inlet has a large tidal prism the first possibility is the most probable. If the tidal prism is smaller the inlet most likely closes.

Fig. 18 depicts the relationship between the ratio p and the ratio $\frac{A}{A_{sm}}$ where A_{sm} is the stable inlet cross section corresponding to τ_{sm} for the bottom material in question. If p is close to unity a stable condition may exist, as long as $\frac{A}{A_{sm}}$ does not deviate too much from unity. If $\frac{A}{A_{sm}}$ increases considerably, for example, because of decreasing littoral transport, the inlet is developing towards a non-scouring condition. If on the other hand $\frac{A}{A_{sm}}$ decreases, for example, because of excessive littoral transport to the channel shoals, a half-moon shaped bar or shoal (Fig. 3) may result which will carry part of the littoral drift across the inlet. This situation could in turn finally result in closing of the inlet.

Referring to Figures 17 and 18, the most stable inlets, needless to say, are those with a very predominant tidal transfer and a p-value close to 1. As soon as p decreases, a usually rather unstable sea shoal or bar develops. The inlet gorge is then subject to fluctuation caused by changes in the offshore bar. While the non-scouring channel, which represents an asymtotic condition, must be classified as "stable" the inlet which by its shear stress is located between a stable channel with $\frac{\tau}{\tau_{sm}} \sim 1$ and a non-scouring channel ($\tau \sim \tau_c$) is usually moving towards a more stable condition either close to or identical with a non-scouring channel or towards a condition with $\frac{\tau}{\tau_{sm}} \sim 1$. A condition characterized by a very small p value can hardly be classified as "stable," although the inlet stays open, but with a reduced and continually changing gorge area the size of which depends entirely upon the material transfer to and the stability of the offshore shoal or bar at the inlet entrance. One may say that the inlet is living on "borrowed time," or as a "boheme" inlet(7).

The ratio $p = \frac{V}{pM_t}$ has been calculated for various inlets in order to check the above mentioned considerations. In most cases M_{net} and not ΣM is known although only in approximation. The assumption, therefore, was made that the littoral transport in one direction was 1.5 times the littoral transport in the opposite direction, which means that $\Sigma M = 5 M_{net}$. V_s is calculated using Kalinske's formula for bed load transport and indicated in Table 13.

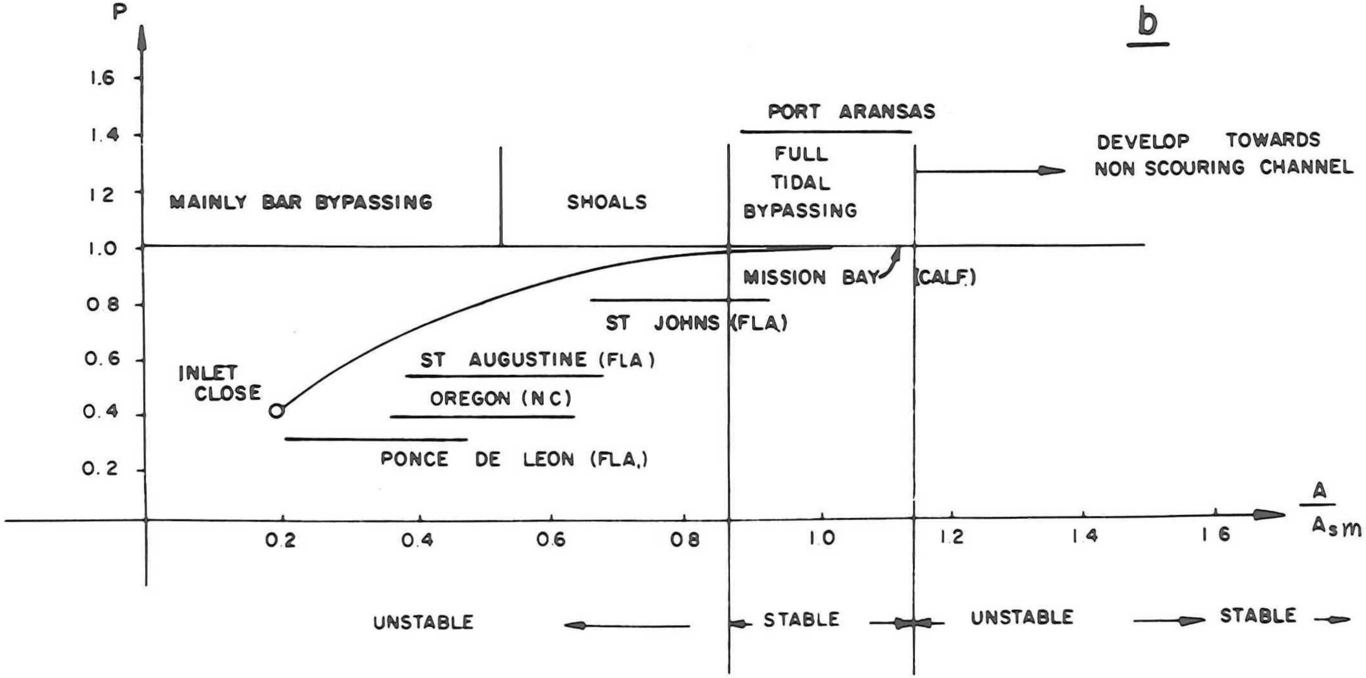

Figure 18. p in Relation to $\frac{A}{A_{sm}}$

TABLE 13
V_s, M_{net} and p-values for some Inlets

Inlet	V_s(cub.yds.)	M_{net} per yr. in 10^6cub.yds.	p
Oregon (N. C.)	3200	1.0	0.4
Calcasieu (La.)	1800	0.1	1.3
Pt. Aransas (Tex.)	2100	0.1	1.4
Mission Bay (Cal.)	750	0.1	1.0
St. Johns (Fla.)	1610	0.3 (0.6)	0.8
St. Augustine (Fla.)	1000	0.3 (0.5)	0.5
Big Pass (Fla.)	540	0.1	0.7
Ponce De Leon (Fla.)	1040	0.5	0.3
Gasparilla (Fla.)	1200	0.1	1.5

The p-values obtained vary between 0.3 and 1.5 and seem to describe the actual inlet stability fairly well. The figures indicated for the St. Johns (Fla.) and St. Augustine (Fla.) Inlets are reduced figures considering the effect of the long jetties installed. The figures in the parentheses are the true figures disregarding the existence of these jetties.

That some p-values are above one may be a result of the inadequacy of the assumptions made as well as Kalinske's bed load formula as applied in this case. The results are of indicative value only.

To evaluate the actual transport rate of littoral materials and their pattern, modern tracing techniques may be helpful. It should be possible to obtain a fairly accurate value for the ratio of drift in two directions any time by measuring concentrations and travel distances in two directions. Such tracer studies are planned to be carried out in the Palm Beach County.

RELATION BETWEEN BOTTOM SHEAR STRESS, CROSS SECTIONAL AREA AND SEDIMENT TRANSPORT AS BED LOAD AT COASTAL INLETS

Kalinske's bed load transport formula(6) is given by the following dimensionless equation:

$$\frac{q_s}{d\sqrt{\frac{\tau}{\rho}}} = 10\left(\frac{\tau}{d\gamma(S_s-1)}\right)^2 \tag{1}$$

$$q_s = \frac{10 \cdot d \cdot \tau^{5/2}}{\sqrt{\rho d^2 \gamma^2 (S_s-1)^2}} = \alpha\tau^{5/2} \tag{2}$$

$$\alpha = \frac{10}{\sqrt{\rho d \gamma^2 (S_s-1)^2}} \tag{3}$$

Considering the inlet bed transport q_s (per unit width) is also equal to

$$\frac{pM_t}{W} = \frac{pM_tD}{A} \tag{4}$$

where $A = W \cdot D$ (W = width, D = mean depth)

One has: $$\alpha\tau^{5/2} = \frac{pM_tD}{A} \tag{5}$$

$$A = \frac{pM_tD}{\alpha\tau^{5/2}} \tag{6}$$

For $\tau \sim \tau_c$ $pM_t \longrightarrow 0$ and A approaches the value

$$A_c = \frac{Q_{max}}{C_c \sqrt{\tau_c/\rho g}} \tag{7}$$

where Q_{max} is the maximum flow passing through the inlet gorge with velocities not causing shear stresses above τ_c and C_c is the corresponding Chezy's friction coefficient.

Shield's bed load transport formula(6) has the advantage of clearly stating that bed load transport = 0 for $\tau = \tau_c$

$$\frac{q_sS_s}{qS_e} = 10 \frac{(\tau-\tau_c)}{\gamma(S_s-1)d} \tag{8}$$

$$qS_e = D \cdot C \sqrt{DS_e} \cdot S_e \tag{9}$$

$$\tau = \rho g D S_e \tag{10}$$

$$q_sS_s = \frac{10\ (\tau-\tau_c)\ C\tau^{3/2}}{(S_s-1)d\ (\rho g)^{5/2}} \tag{11}$$

$$q_s = \frac{10(\tau-\tau_c) \cdot C\ \tau^{3/2}}{(S_s-1)d\ S_s \cdot (\rho g)^{5/2}} \tag{12}$$

$$q_s = \tau^{3/2}(\tau-\tau_c) \cdot \frac{10 \cdot C}{(S_s-1)\ d \cdot S_s(\rho g)^{5/2}} \tag{13}$$

$$q_s = \tau^{3/2}\ (\tau-\tau_c) \cdot \beta \tag{14}$$

$$\beta = \frac{10C}{(S_s-1)\ d\ S_s(\rho g)^{5/2}} \tag{15}$$

$$\beta = \beta' \cdot C \tag{16}$$

$$\beta' = \frac{10}{(S_s-1)\ d\ S_s(\rho g)^{5/2}} \tag{17}$$

$$q_s = \frac{pM_t}{W} \tag{18}$$

$$q_s = \frac{pM_tD}{A} = \tau^{3/2}\ (\tau-\tau_c)\beta'C \tag{19}$$

or

$$A = \frac{pM_tD}{C\beta'(\tau-\tau_c)\tau^{3/2}} \tag{20}$$

Assuming, introductorily, that pM_t is constant, and that D and β are also constant or vary little. It is self-explanatory that Eq. (20) is not valid up to $\tau = \tau_c$. Considering a normal case where the inlet bottom is covered by sand of medium grain size about 0.2 mm: Putting $\tau = \tau_{sm}$ in Eq. (20) for $A = A_{sm}$ and τ_c = about 1/6 τ_{sm} (a practical, rather than a limiting value) one has:

$$A = \frac{K}{C\beta'} \frac{1}{(\tau - \frac{\tau_{sm}}{6})\tau^{3/2}} \tag{21}$$

$$K = pM_tD = \text{constant} \tag{22}$$

Fig. 19 depicts Eq. (21), putting $\tau = \tau_{sm}$ for $A = A_{sm}$ and $\frac{K}{\beta'} = 1$, while the value of C varies and may decrease to approximately half of its value when bottom changes from rippled to duned and plane.

One has

$$A \sim \frac{1}{C}\frac{6}{5}\ \tau^{-5/2} \tag{23}$$

or

$$CA \sim 1.2\tau^{-5/2} \tag{24}$$

It will be seen that this relation demonstrates a very strong increase of A with decreasing τ. If pM_tD followed a similar exponential relationship, A would stay constant, but it is most unlikely that pM_tD would develop in that way.

The dotted line in Fig. 19 demonstrates an attempt to introduce the friction factor in Eq. (21). For the rippled bottom occurring for low velocities (1-2 ft/sec) C is assumed to be as low as 25 m 1/2/sec, (considering the dotted curve valid for a shallow inlet). For the high velocities (3-4 ft/sec) C = 50 m 1/2/sec. Using the full-line diagram it may be seen that A for $\tau_s = 0.25$ kg/m^2 is six times A for τ_s = kg/m^2. If equilibrium according to Eq. (21) should be re-established pM_tD had to be six times less too. Unless the inlet is improved by very

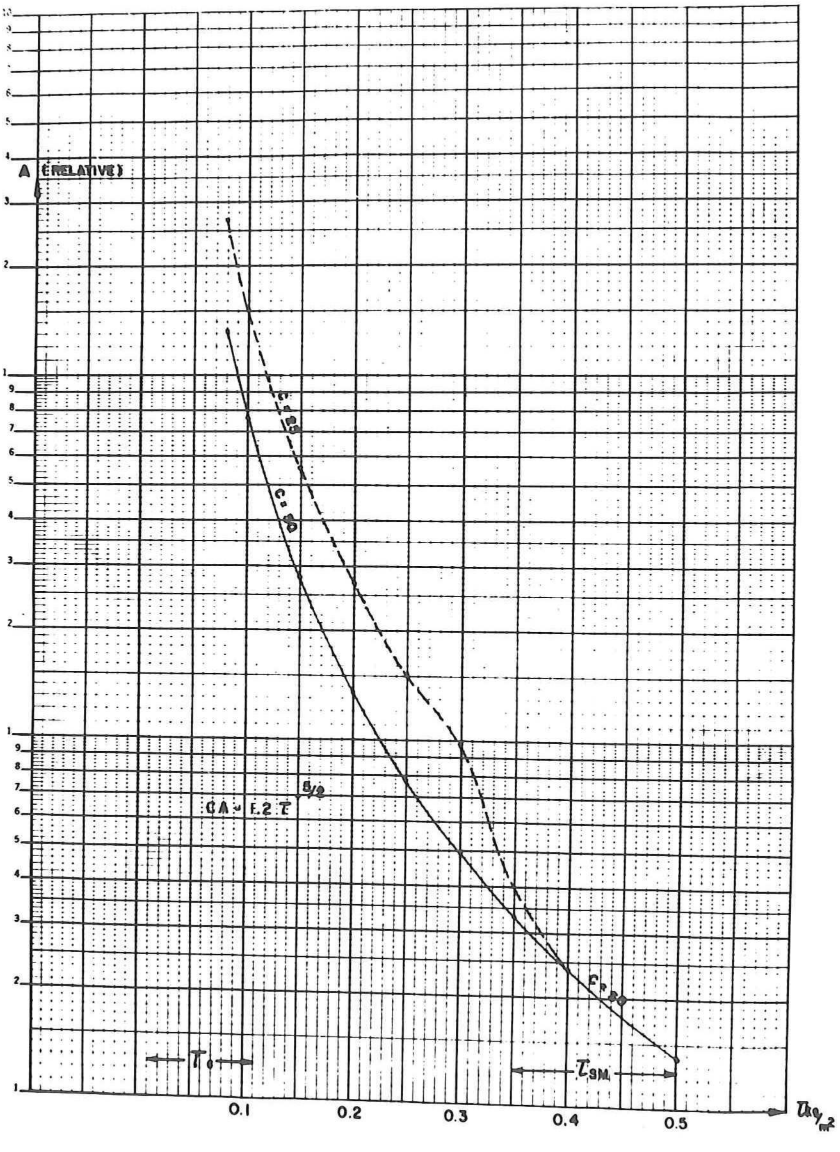

Figure 19. Relationship between A, τ, and C

long jetties, eliminating almost all transport of littoral material to the inlet, this seems to be a very unlikely situation. A more practical case would be that pM_tD was reduced to say half of its value by the erection of jetties of reasonable length. This assumes some continued maintenance dredging in the inlet channel, as it is almost always the case. τ_{sm} may then drop from ab. 0.5 kg/m^2 to somewhere between 0.30 kg/m^2 and 0.40 kg/m^2, because we may have moved from the heavy load to the medium load or light load area. This development needless to say is an advantage to navigation, because cross section increases and peak velocities drop somewhat.

Fig. 19 explains why the non-scouring channel represents a "theoretical limit case" which has the character of "an open bay" rather than a channel (San Diego Harbor, New York Bay entrance, and Chesapeake Bay). A is very large and τ drops to a value close to τ_c.

Considering a cross sectional area A = W·D, experience from tidal inlets, as mentioned by Bruun(9), demonstrates that $A \sim Q^{0.95}$, $W \sim Q^{0.71}$ and $D \sim Q^{0.24}$. Increases or decreases of the cross sectional area will mainly reflect itself in changes in W and only in small changes in D.

In practice, the pM_t varies within a limited range. With respect to variation in pM_t the figures in Table 13 will serve as a guidance or indication only. Combining the very limited possibilities for variation of the nominator of Eq. (21) with the strong variation in the denominator makes it understandable why inlet stability is so sensitively related to a narrow band of τ_s values. Stability of a tidal inlet in sand material (considering a certain period of time - the order of magnitude must be at least 50 to 100 years) actually does only seem to exist for $\tau = \tau_c \pm \epsilon$ and for $\tau_s = \tau_{sm} \pm \epsilon$, where ϵ is a relatively small value perhaps not exceeding about 15% of the average τ_{sm} for fine sand material (about 0.2 mm), excluding at this time any kind of cohesive material as well as material size above fine sand (> ab. 0.5 mm).

CONCLUSION REGARDING INLET STABILITY

With reference to the above-mentioned, it is therefore quite natural to explain the tidal inlet stability phenomena as a result of "the inlet's being bothered by material from the adjoining shores." The reaction of the inlet to this situation is that its cross sectional area attains such dimension that currents concentrate enough to produce the necessary high velocities and thereby shear stresses to flush the inlet for the surplus material which poured into it from the sides: The actual size of the shear stresses necessary to produce the "desired effects" depends in detail upon inlet geometry, inlet material, and upon the concentration and magnitude of littoral material transferred to the inlet as bed load and as suspension load. The cross section then adjusts itself to the actual combination of inlet currents and impact of factors from outside influencing the stability of the cross section. This adjustment is related to a rather narrow range of shear stresses, which as explained below are located in a transition zone, with respect to bottom roughness, where nature's waste to friction losses are at a minimum and where relatively smaller adjustments in τ_{sm} will be able to handle "the occurring pM_t situation", that means flush the inlet for surplus material.

It will be of interest to consider the circumstances in detail which may have given rise to a τ_{sm} varying from about 0.35 kg/m^2 to 0.50 kg/m^2 only. These shear stresses correspond to velocities of approximately 0.8 to 1.0 m/sec. (ab. 3 to 4 ft/sec).

This interesting subject will be dealt with by the author in a forthcoming article under the auspices of the Tidal Hydraulics Committee of the American Society of Civil Engineers, referring to the experience from rivers and streams where the development of bottom geometry from rippled, duned to plane, and phenomena associated with the anti-dune reveals that the transition zone from dune to plane bottom by which friction decreases (Fig. 20, Reference 10), and sediment transport increases (Fig. 21, Reference 12), because the shear stress is now exerted upon the entire bottom area, must be the tidal inlet's "instrument to handle its problems".

Another peculiarity as revealed by Fig. 22 (Reference 14), is that bed load transport for velocities of approximately 3 ft/sec will stay consistent with fine sand regardless of very strong variations in depth. Nature, through its 1 m/sec (ab. 3 to 4 ft/sec) policy for tidal inlets in alluvial material, seems to have chosen a simplified and economical approach to solution of its problems and - most surprisingly - it even seems to have foreseen the metric system!

BACKPASSING

Backpassing is a procedure by which material which eroded from the beach and deposited in the offshore waters or in other ways, for example, by inlet currents, was carried out to deep waters is brought back to the beach again.

The main problems involved in backpassing include the location of the proper sources of sand, suitable for nourishment, in the offshore areas; bringing this material to shore economically; and finally, the development of equipment needed for dredging in offshore waters and for discharge of the material where it is needed without rapid losses of it to deeper waters.

At present, the U. S. Army Corps of Engineers are carrying out considerable surveys to locate offshore sources (16), and results from the Florida Atlantic Shores seem promising.

With respect to the development of special dredging equipment, which is able to dredge material in offshore waters and transfer it to the beach, reference is made to paper by A. L. McKnight printed in these Proceedings (Reference 13).

D-6 STUDIES OF FLOW IN ALLUVIAL CHANNELS

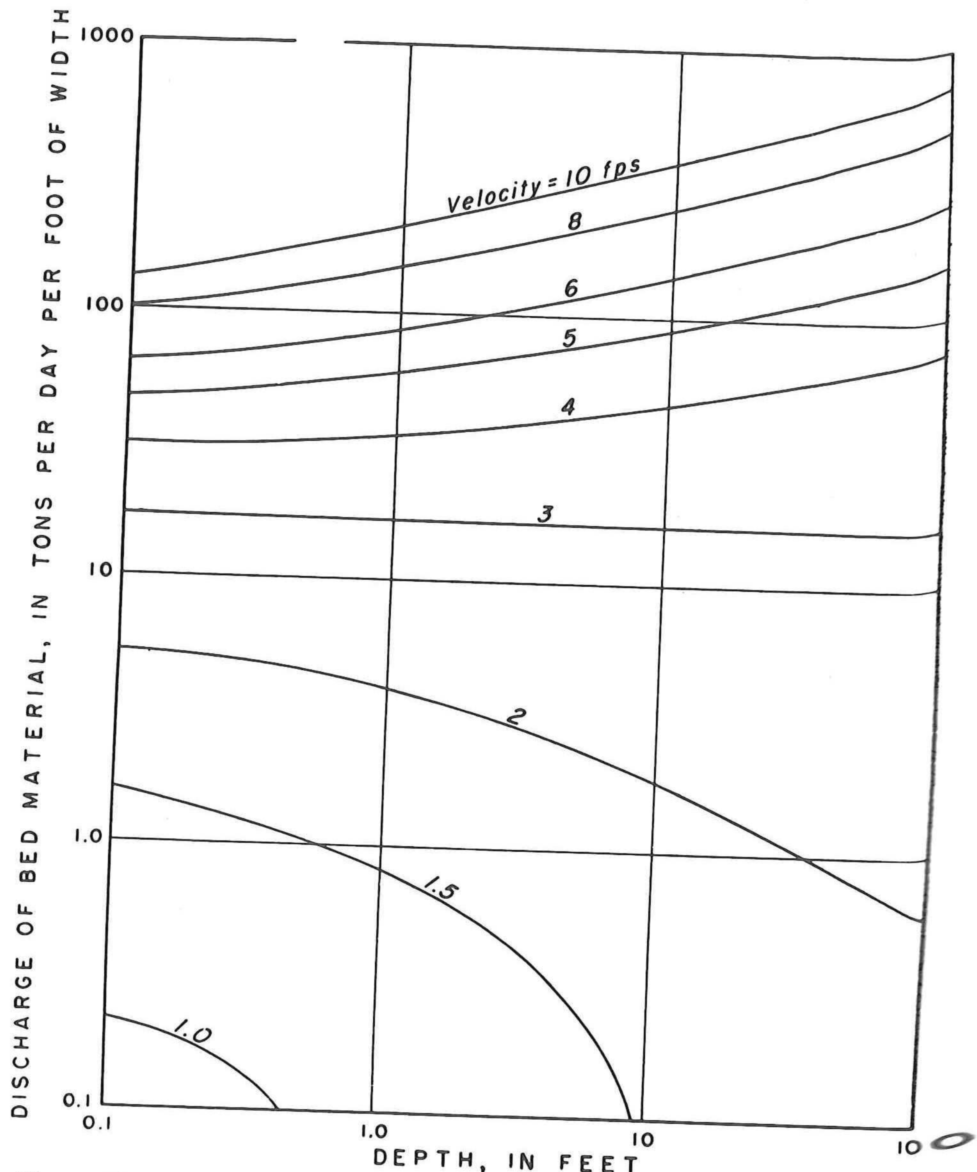

Figure 20. Effect of Depth on the Relationship between Mean Velocity and Empirically Determined Discharges of Bed Material (0.3mm Medium Diameter) at 60° F. B. R. Colby, 1961

LAURSEN AND ZERNIAL ON ALLUVIAL CHANNELS

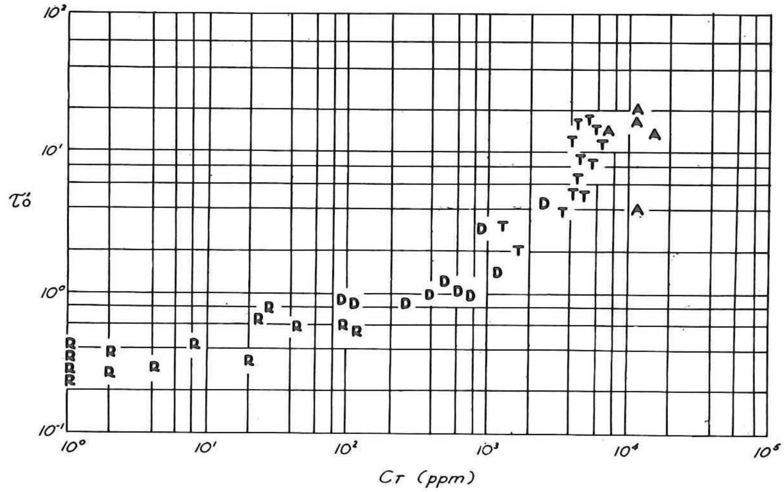

Figure 21. Concentration versus Particle Shear. B. M. Laursen and G. A. Zernial, 1962

STUDIES OF FLOW IN ALLUVIAL CHANNELS

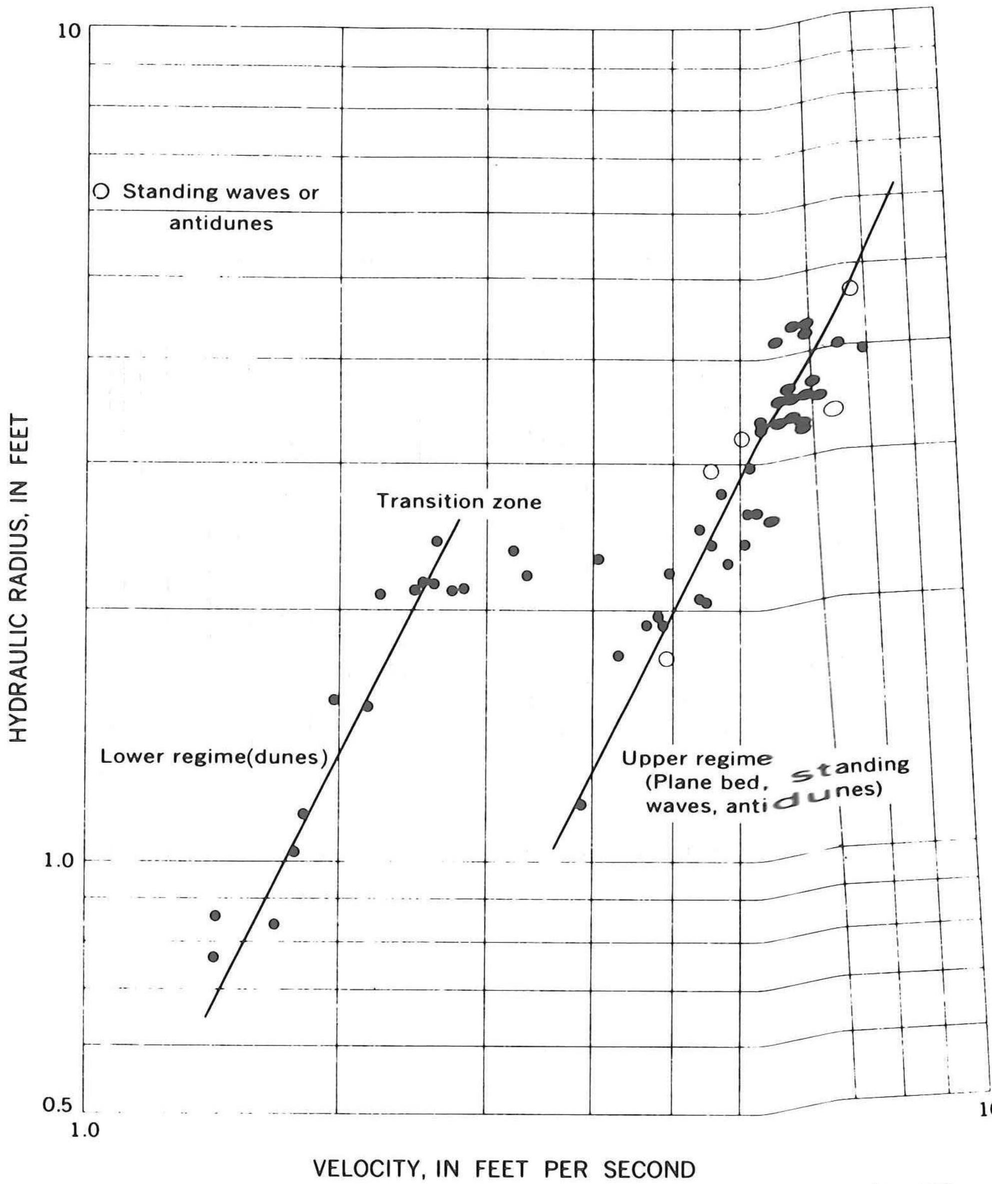

Figure 22. Relation of Hydraulic Radius to Velocity for Rio Grande near Bernalillo, New Mexico, C. F. Nordin, 1964

Various "en miniature" measures on by-passing have been used earlier. Fig. 23 shows "artificial nourishment" by a bulldozer operating in the uprush zone (Deerfield Beach, Lower East Coast Florida, March 1962). It is clear that the benefits of such operation are of a very temporary nature, but it is still justified in emergency cases and as a measure to create a temporary protection for the toe of eroding dunes. This method, therefore, is being used to some extent on the Florida Atlantic coast to buffer against northeast storms, that is, at Fernandina Beach and at various beaches on the southeast coast of Florida.

An offshore scraper was built in England 6 to 8 years ago to the design of Mr. R. C. H. Russell, Hydraulics Research Station, Wallingford, to test a new method of building up beaches by bringing sand in from offshore. It is a type of scraper on broad wheels that is moved to and fro, between a system of offshore anchors and a three-drum excavator on the beach. The scraper is operated by the manipulation of three wire ropes: one hauls it inshore, one drags it offshore, and the third controls the position of the rotatable bucket.

The machine was of 1 cub.yd. capacity and was looked upon as a half scale model of a machine of 8 cub.yds. capacity. No fundamental defects were found in the machine, but the development of the full-size machine was held up for the lack of any suitable prime mover having three precisely controllable drums each capable of holding 600 ft. of 1 inch diameter wire rope.

Similar scraper experiments are at this time being carried out on the Florida Atlantic shore where the Town of Jupiter Island has engaged in an experimental method of supplying sand for beach nourishment. The Town of Jupiter Island is a unique resort area composed of approximately 250 winter residents. Their residences have been threatened with destruction over the past several years along the approximately four-mile strip of the inhabited area of Jupiter Island; the island itself being approximately 15 miles long.

Since 1957, the town has relied on pumping sand from the Inland Waterway, west of the island, to the beaches. Approximately 700,000 cub.yds. of sand has been pumped since that time. It is realized that the supply of available material from the Inland Waterway will soon be exhausted, so other means of supply beach nourishment are being investigated. As a result of suggestions made by one of the residents, who was formerly in the mining business, the conception of mining sand from the ocean and placing same in the surf area on the beach was proposed.

In June, 1963, the town entered into a contract with Dickerson, Incorporated of Stuart, Florida. Basic equipment involved is as follows: (1) Three-drum Sauerman drag scraper unit powered by Model S-110 diesel motor; (2) a 3-cub.yds. bottomless crescent drag bucket; and (3) offshore anchor arrangement which was devised by the contractor.

The equipment was ordered in 1963 immediately upon the placing of the contract, however, a delay in the delivery of the equipment

Figure 23. Artificial Nourishment by Bulldozer at Deerfield Beach, Florida (1962)

prevented substantial operations during 1963. Unseasonable rough surf conditions halted operations. Seas at that time reached heights of 8 to 10 ft. and the anchor barge was sunk on September 24.

The contractor moved approximately 10,000 cub.yds. of material during the 15 days of operation. Five days of this period were ten-hour daytime shifts only. During the final days, the contractor began a ten-hour night shift. The experience gained during the short period of operations paid dividends. Fig. 24 shows the 3 cub.yds. bucket on its way back to shore at Jupiter Island.

Very rough weather in April and May of 1964 prevented the beginning of operations until May 16, 1964. As a result of the experience gained during the 1963 operations, the barge anchor arrangement was changed so that the shifting of the tail anchor barge could be accomplished from shore much more efficiently. The anchor arrangement is set approximately 900 ft. from shore, and the excavation is being made in an approximate 600-ft. arc in a borrow area known as Zone 2 (depth 10 to 12 ft.) the centre of which averages some 750 ft. perpendicular to the foreshore. During the period May 16th to September 3rd a total of 125,000 cub.yds. was produced and left in two stockpiles.

The results of the tests run from June to September, 1964, at Jupiter Island, with two borrow areas, Area I and Area II, located about 2,000 ft. apart (Area I North, Area II South) may be summarized as follows:

Hydrographic Surveys have demonstrated that the depths of the two borrow pits developed as indicated by Figs. 25 and 26 (Area I) and Fig. 27 (Area II). Comparison between the April survey (before operation started) and the June survey for Area I (Fig. 25) shows the steepening of the offshore bottom, and the narrow trench dug by the bucket (Fig. 24). Comparing the June, September, and October surveys, it is evident that material moved in the borrow pit from south because of the predominant northward drift during July and August. The dragline moved to Area II about July 1st. The October 6 survey shows that little trace of the borrow pit is left. The 14 and 16 ft. depth contours, however, indicate that the bucket ditch connecting the borrow pit with the beach may have caused a rip current. The nearshore bottom steepened about 2 ft. up to 15 ft. depth or more. This, however, may be a seasonal phenomena.

In Area II, (located 2,000 ft. south of Area I) scraping started about July 1st, 1964, and operation was stopped at the beginning of September because of the hurricanes. The October 6 survey compared to the June 29 survey demonstrates that material from the sides, apparently mostly from the north side, drifted down in the borrow pit. This may be the result of the 1964 hurricanes. (Mainly Cleo and Gladys.) Drift in this period should normally be predominantly northward.

Remnants of the borrow pit are still visible on the October 6 survey where all depth contours between 7 and 13 ft. are bent towards the shore. The 14, 15, and 16-ft. depth contours demonstrate a local

Figure 24. Sauerman Scraper in Operation, Jupiter Island, Florida

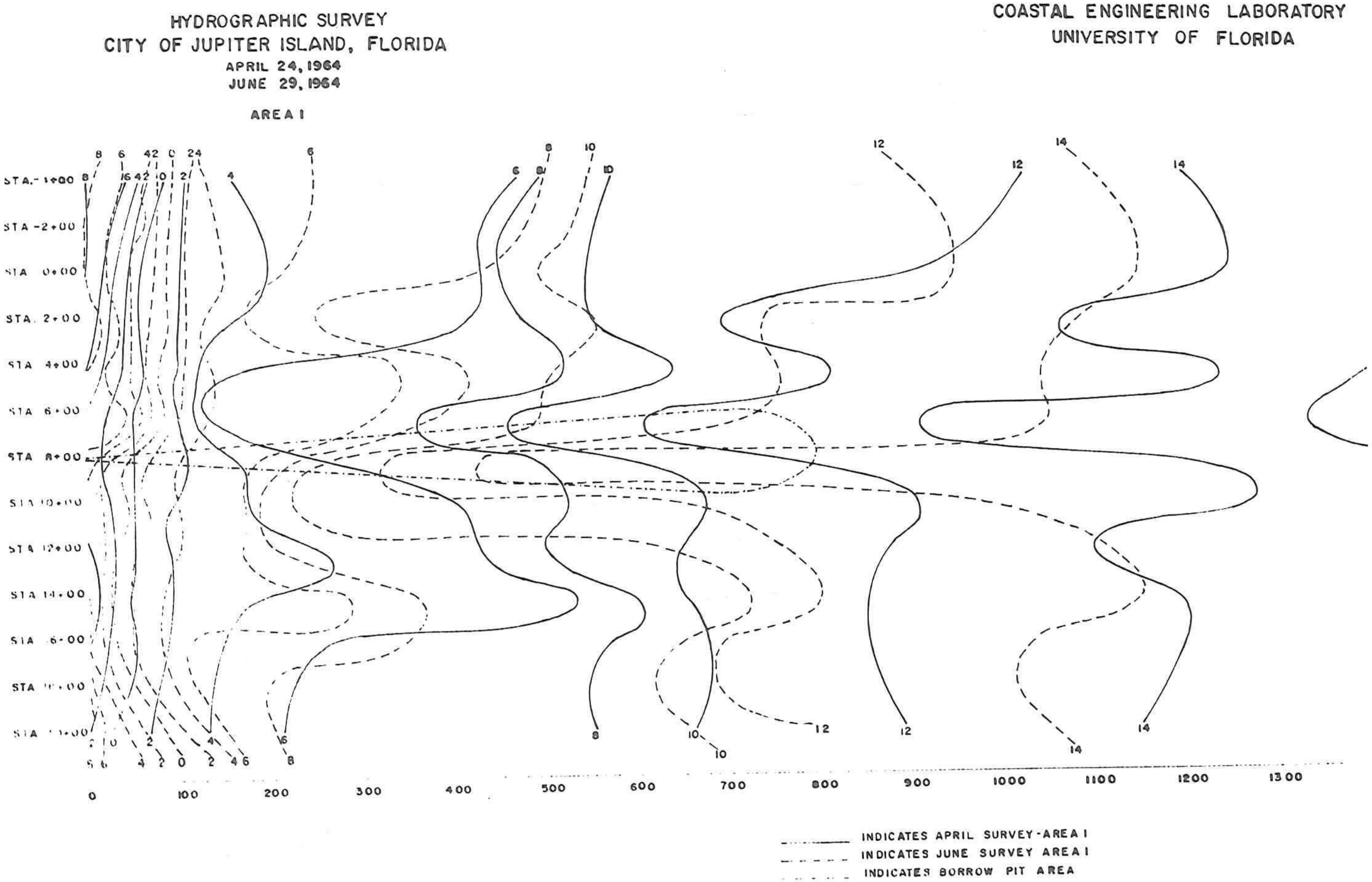

Figure 25. Development of Borrow Pit, Area I, Jupiter Island, Florida

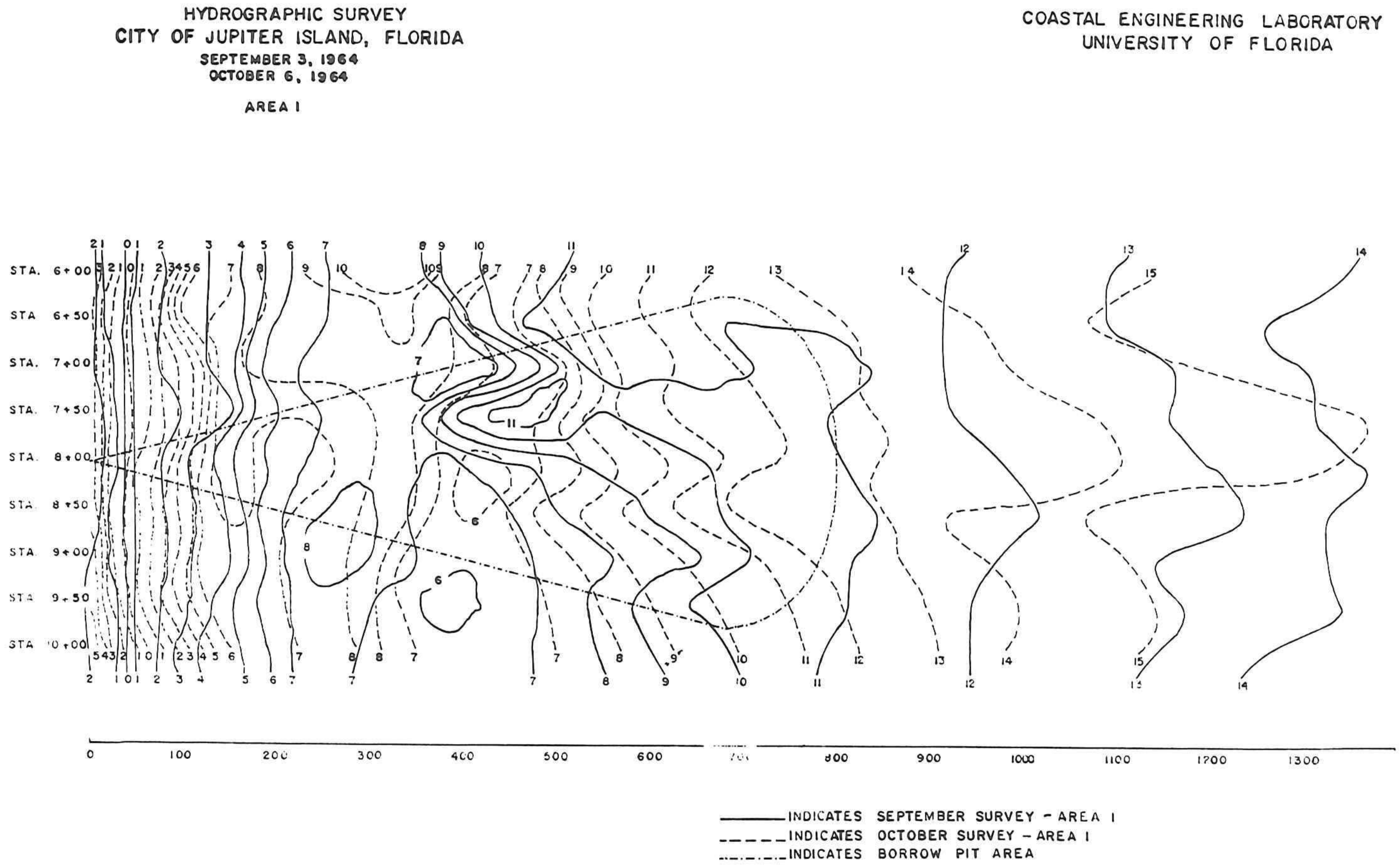

Figure 26. Development of Borrow Pit, Area I, Jupiter Island, Florida

outward bent curvature which could be the result of a rip current caused by the bucket ditch. The borrow pits are not to be seen in the bottom topography secured by the January, 1965 survey.

Scraper tests were carried out by dumping of tracers between the borrow pit and the shore (50 to 120 lbs.) in order to see if the 3 cub. yds. bucket picked up its material where it was supposed to, in the borrow pit, or if it dropped material on its way to shore and picked up new material closer to shore, a less advantageous situation.

The tests for Area I demonstrated, as also clearly revealed by Fig. 25, that the bucket dug a trench when passing over the nearshore bottom. Some material picked up from the borrow pit was lost here and replaced by material from the trench. The same was true for Area II but to a less extent. In neither case did the amount of "nearshore fill" build up to any quantity of importance. The contractor has been aware of this problem and took the necessary measures on his own initiative as soon as it was realized by him.

Borrow Pit Tests carried out by dumping of tracer material around the borrow pit of Area I, demonstrated that material from all sides "sloughed" down in the borrow pit. It was, however, also observed that material from the south and from the inside (west) migrated across the borrow pit, partly towards the shore and partly in offshore direction. Wave action was very weak between the time of dumping and sampling so the material movement which occurred should probably be interpreted as a result of some tidal current action combined with the stirring-up of bottom material by the bucket which created a certain concentration of suspension material which as a density current moved partly longshore as it was often visible during the scraper operation and partly offshore along the bottom perhaps assisted by some rip currents caused by the borrow pit and trench at the same time as some coarser material probably was pushed shoreward by the action of low swells.

No borrow pit tests were run for Area II because it was decided to let material from scraper tests and littoral drift tests on the beach provide the tracers for the transversal drift. From the samplings in the borrow pit and between the pit and the shore comes forth that little beach tracers (beach material) ended up in the borrow pit. As explained below, it apparently migrated longshore on the beach in a northward direction because of the prevailing wave action during the summer season. Some tracers dumped inside the borrow pit migrated in an offshore direction, but the bucket may have been responsible for that because it, upon its back movement, pushed some material in the offshore direction. "Density currents" and rip currents as mentioned above may have had a similar influence.

Littoral Drift Tests. Tracer material was dumped on the beach in both areas. Sampling revealed that some material during the summer season by southeast wave action migrated northward from Area II where it reached or passed Area I (2,000 ft.). Winter storms or perhaps the September 1964 hurricanes dragged some of it out in the Area I borrow pit. It is possible that a similar exchange of material took

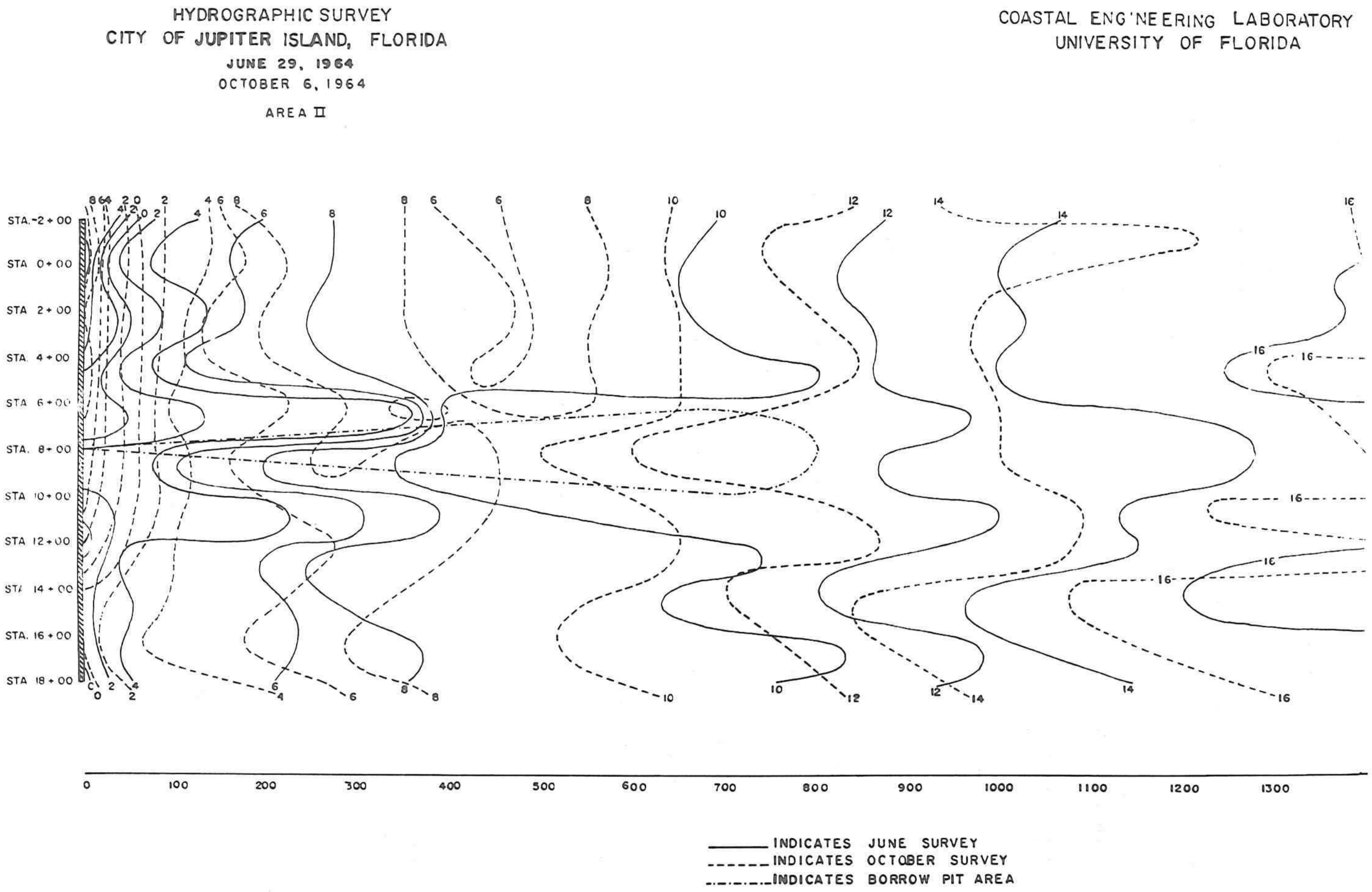

Figure 27. Development of Borrow Pit, Area II, Jupiter Island, Florida

place between Area I and Area II in the way that material in Area I was first washed out to 5 to 12 ft. depth during fall and winter storms, after which it traveled southward until it passed the borrow pit of Area II. There is, however, no direct evidence of such movement although it is clear that red tracers injected in the beach spoil were transferred to the borrow pit and to areas of the bottom, seaward of the borrow pit.

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