CHAPTER 43

CASE HISTORY OF
MISSION BAY INLET
SAN DIEGO, CALIFORNIA

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ABSTRACT

The Mission Bay Inlet was designed as a "Non-Scouring" entrance channel by the Los Angeles District, Corps of Engineers, in 1946. Construction of the inlet was completed in 1959 and the entire project was completed in 1963. A channel with over twice the cross-sectional area required by the "O'Brien" equation was developed to reduce the average cross-sectional tidal currents to less than two feet per second. The design depth of -20 feet MLLW eliminated bottom movement induced by wave action- except during the most severe storms. The jetties are sealed to the +4 foot elevation and extend to the -25 foot depth almost entirely eliminating the intrusion of littoral drift. The channel has shoaled at a rate of less than 20,000 cubic yards per year since final dredging in 1959, indicating the soundness of this concept. This case history was prepared under contract to the Coastal Engineering Research Center, U. S. Army, Corps of Engineers, and project data and aerial photographs were obtained from the Los Angeles District, U. S. Army, Corps of Engineers, and the City of San Diego.

HISTORY

Mission Bay, prior to 1946, was a natural estuary of over 4,000 acres (Photo 1). The major drainage feature into the estuary was the San Diego River with a drainage area of 435 square miles. San Diego is a semi-arid area and the river is normally dry, but during severe floods it may flow in excess of 100,000 cubic feet per second and carry large quantities of silts, sands, cobbles and floating debris. There has not been a flood of this magnitude since 1927. The San Diego River originally discharge into either San Diego Bay or into the southeast corner of Mission Bay. The latter entrance to the ocean was a natural inlet through a sandy beach. While the thread of the inlet meandered constantly, it generally had a controlling depth of 6 feet mean lower low water. The tidal currents in the natural inlet could exceed 4 knots and were sufficient to counteract the force of waves and infringement of littoral drift and maintain a permanently open tidal inlet.

It was realized that if the San Diego River were allowed to continue to discharge into San Diego Bay, serious shoaling would result and would interfere with commercial shipping. In 1876 a permanent levee was constructed, protecting San Diego Harbor. From 1876 to 1946 the river was permitted to discharge its debris into the "useless" Mission Bay.

DESCRIPTION OF AREA

Geomorphology

     Indian legend has it that Pt. Loma was originally an island.
This is undoubtedly true.   There is also little doubt that a few
hundred years ago there was much more rainfall than at present and
the San Diego River supplied a great deal more sediment to the coastal
plain. However, the history of the river up to 1876 was alternate
discharge into either San Diego Bay or Mission Bay, or False Bay
as it was originally called.

Littoral Drift

     Studies by the Beach Erosion Board of the Corps of Engineers
in 1942(2) and 1949-50(3) indicate that the Mission Bay area is
essentially a beach compartment extending from La Jolla on the north
to Point Loma on the south.   The San Diego River, originally dis-
charging through Mission Bay, is the principal source of beach sand.
It was observed that while there was some upcoast sand movement
in the summer and downcoast in the winter the area was essentially
in littoral balance.   They also found in their 1950 studies that
there was a very positive onshore-offshore movement from summer to
winter.

Tides

     At the entrance to Mission Bay the mean range of tide is 3.8
feet and the diurnal range is 5.4 feet.   The extreme range varies
from 2.5 feet below to 7.0 feet above mean lower low water.   Mean
sea level is 2.8 feet above MLLW.   The tides are characterized by
a diurnal inequality and, as the maximum run is from higher high
tide to lower low tide, inlets in southern California generally have
higher ebb-tide than flood-tide velocities.   This infers an ability
to move more sediments during periods of ebb tide than during flood
tides.

Waves

     Wave action is generally mild in this area having periods of
6 to 16 seconds and heights of less than 3 feet.   However, waves 18
feet in height can occur at the entrance to Mission Bay and a
significant wave height(H) of 16 feet was used for design of the
jetty structures.

     Also, based upon experience and consideration of the larger
storm waves it was decided to use a navigation project depth of -20
feet MLLW to insure the occurrence of non-breaking waves in the
entrance channel at all times except during the most severe storms.

Winds

     Winds rarely exceed 30 knots, and their main import is how
they affect the maneuverability of boats under sail.

Rainfall

     Average annual rainfall in this semi-arid region ranges from
10 inches along the coast to 38 inches in the mountains. 80% falls
from December to March inclusive and, in the lower reaches, the
San Diego River is dry most of the time. The last major runoff
was in 1927. During peak floods it may run as high as 100,000
cubic feet per second. Thus, while the river has almost no
influence on the tidal characteristics of the bay and its ocean
inlet, the sediments carried during periods of major runoff can
be of large quantity and materially affect the bay and/or the
adjacent shoreline.

ANALYSIS OF DESIGN

Project Authorization (Fig.1)

     The first mention of plans for development of Mission Bay as
a navigation area was in 1941 in studies by the City of San Diego
and the Corps of Engineers to improve the lower San Diego River for
purposes of flood control. As this study progressed it became
evident that maximum benefits could be obtained by a combined
flood control-navigation project.

     The combined project was adopted by Congress in July 1946,
as presented in House Document No. 760, 79th Congress, 2nd Session.
Detailed design and cost estimates were presented by the Corps of
Engineers in "Definate Project Report on San Diego River and
Mission Bay", dated January 1949. The Federal Government was
responsible for the main channel and its sideslopes, the dredging
of the east and west basins, the dredging of the navigation
entrance channel and construction and maintenance of the three
jetties defining the navigation and flood control channels.
Navigation depths of -20 feet MLLW were authorized. The Ventura
Boulevard Bridge and everything inland of it was the responsibility
of the City, and navigation areas were to be dredged to a depth
of -8 MLLW. The Federal portion of this project was essentially
completed in 1959 at a cost of about $10,000,000. Local public
interests have expended about $19,000,000 to date and private
interests over $18,000,000. Ultimate public recreational develop-
ment is expected to exceed $50,000,000 (4).

Design Objectives

     A well conceived harbor, among other considerations, represents
a balanced sedimentation system. It is hopefully a large quiet body
of water with a relatively narrow protected channel to the sea

Figure 1.

requiring minimal maintenance. Due to waves, tides or river discharge, currents of varying rates may be created. If these currents have sufficient velocity, they will either initiate movement and transport sediments or if sediments are already moving, continue them in motion. These currents and resultant sediment movement may cause internal changes in bottom depths or may cause a net gain or loss of sediments to the total harbor complex with resultant shoaling or scouring, either of which may be detrimental.

Engineers in Southern California learned as early as 1876 that if the rivers of this semi-arid country were diverted out of the harbor area and preferably discharged into the sea on the down-drift (littoral) side of the entrance a major source of harbor shoaling could be eliminated. This was demonstrated (successfully accomplished) in 1876 when the San Diego River was diverted out of San Diego Bay and in 1916 when the Los Angeles River was diverted out of Los Angeles-Long Beach Harbor.

The other major exterior source of shoaling is sand from the adjacent beach being carried into the entrance by tide and/or wave induced littoral current, creating shoals either in the entrance or in the quiet water area of the harbor. If the entrance jetties are sand tight, of sufficient elevation, and are extended seaward of the area of littoral drift, most of this type shoaling is eliminated. However, in areas of predominantly uni-directional littoral drift, a sand-bypass system will ultimately have to be established to prevent the impounded sand from being carried over or around the seaward end of the intercepting jetty and thus shoaling the entrance. Jetties extended to the 25 to 40 foot depth contour at Newport Beach in 1936 and Pt. Hueneme in 1941 had demonstrated this concept.

Even though there are no external sources of sediments to cause shoaling of a harbor or its entrance channel and the discharge from the uplands has been diverted, tidal or other currents still are a liability. They may either cause internal movement of sediments (with adverse shoaling or scouring effects) or inter-fere with the safe and efficient operation of boats using the harbor.

These experiences resulted in a concept known as a "non-scouring" tidal channel, i.e., enlarge the inlet cross section to a point where tidal current velocities are reduced below their capability to move bottom material. A secondary benefit in such reduced velocities is the elimination of hazards to boats. Such a maximized cross-sectional area requires a balanced design between width and depth. Increased depth permits deeper draft boats to use the harbor, prevents a breaking wave, or reduces the steepness of waves during the storms, making for a safer entrance. Wave induced bottom movement can also be eliminated, or greatly reduced. The principal effects from increased width of channel is less boat traffic congestion and greater maneuver area for sail boats. On the minus side are increased dredging costs, sometimes more costly jetties, increased land acquisition costs and the introduction of more wave energy into the inner harbor to the possible detriment of berthed boats and interior land perimeters.

BASIS OF INLET DESIGN 1946 to 1949

1.   The design relationship between the dimensions of a harbor
or bay and its channel connection to the ocean at this time was
largely an emperical one.   The most acceptable work in this field
had been done by Lt. Col. T.M. Robins of the Corps of Engineers(6),
and Morrough P. O'Brien of the University of California at
Berkeley (5).   They had developed the following relationships
based strictly on observations of existing harbors, bays and
estuaries.

     In 1931 O'Brien developed the equation $A = 1,000 \ T^{0.85}$
where A = area of the entrance section below mid-tide in square
feet.   T = volume of the tidal prism in square-mile feet between
MLLW and MHHW, and has a value ranging between 7.0 and 3000.   In
1933 Robins published the relationship developed by C.I. Grimm
of his staff as $A = T$ where A = Area of channel cross-section at
mid-tide in square feet and T = The Tidal prism in acre-feet
between MLLW and MHHW.

2.   Mission Bay in its natural state reportedly had a tidal prism
of about 10,400 acre feet (Photo 1).   The minimal cross-sectional
area of the entrance channel as surveyed in 1947 was 6000 square
feet (Fig.2a).   Compared to this field data, the Robins' equation
shows a computed equilibrium entrance area of 10,400 sq.ft. while
the O'Brien equation shows an area of 9,100 sq. ft.   The reported
10,400 acre-feet of tidal prism may be in error or, if correct,
the very flat slopes of the marshlands and the constructions of
tidal flow at the two Ingraham Boulevard bridges may have reduced
the resultant required channel cross-section from the O'Brien
derivation of 9100 sq. ft. to the actual 6000 sq. ft.

3.   The designers of Mission Bay increased the cross-sectional
area of the planned entrance channel to 19,800 square feet below
mean tide level and even though creation of the Aquatic Park
resulted in some 30,000,000 cubic yards of dredging, the net
result was a tidal prism of about 9,200 acre feet.   However, the
flat slopes of the marshland areas were eliminated and there is
very little frictional or time loss in disposing of the tidal
prism.   Thus, using the O'Brien equation, the jettied entrance
channel computes at an equilibrium flow area of 8900 square feet
as compared to the design area of 19,800 sq. ft.   Treating the
tidal current produced from a spring tidal drop of 5.4 feet in
6 hours, average channel velocity would be 0.97 ft. per second.   It
was felt that maximum velocity would not exceed 2 ft. per second
except during spring tides, and scouring velocity along the bottom
of the channel should be even less.

MISSON BAY INLET

Station 40+00

FIGURE 2a

Station 48+00

FIGURE 2b

MISSION   BAY   INLET

Station 30+00
FIGURE   2 c

Station   13+00
FIGURE   2 d

JULY 1954

AUG. 1955

JULY 1958

JAN. 1959

MAY 1970
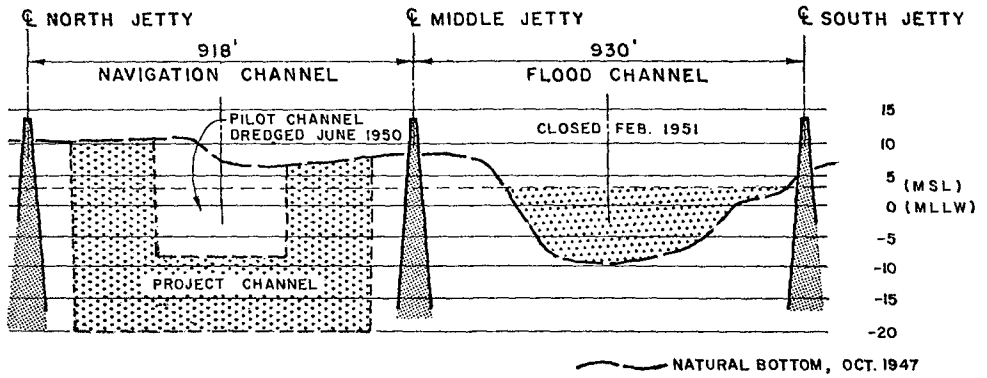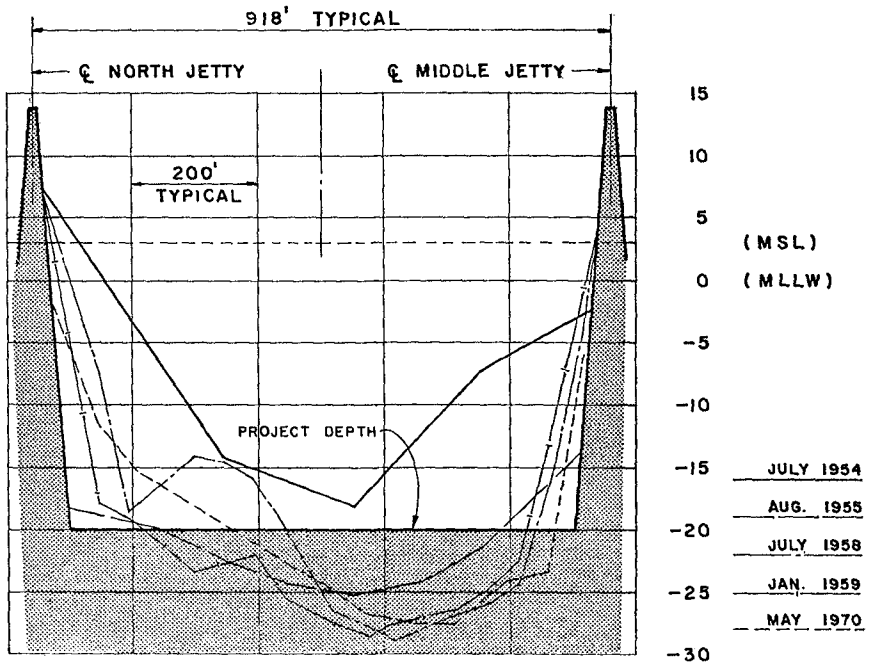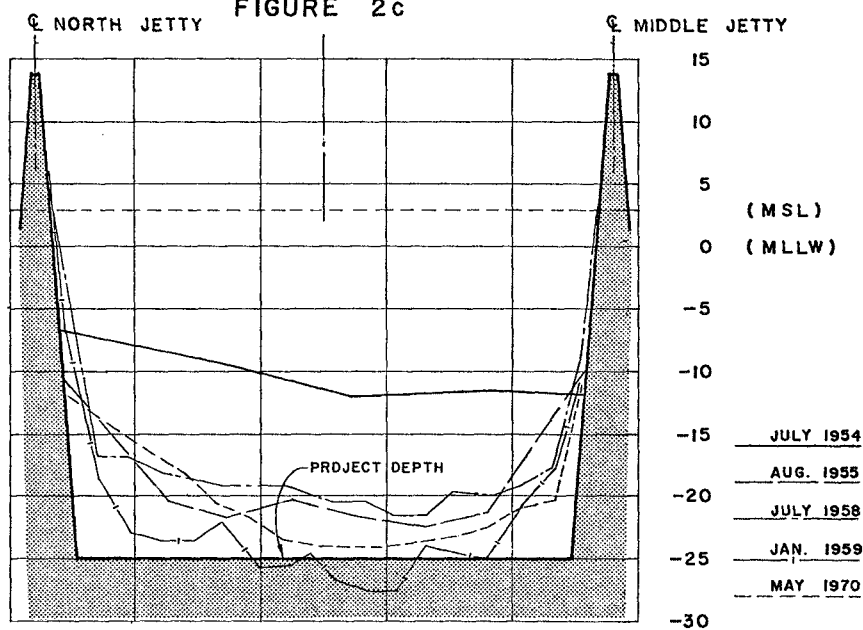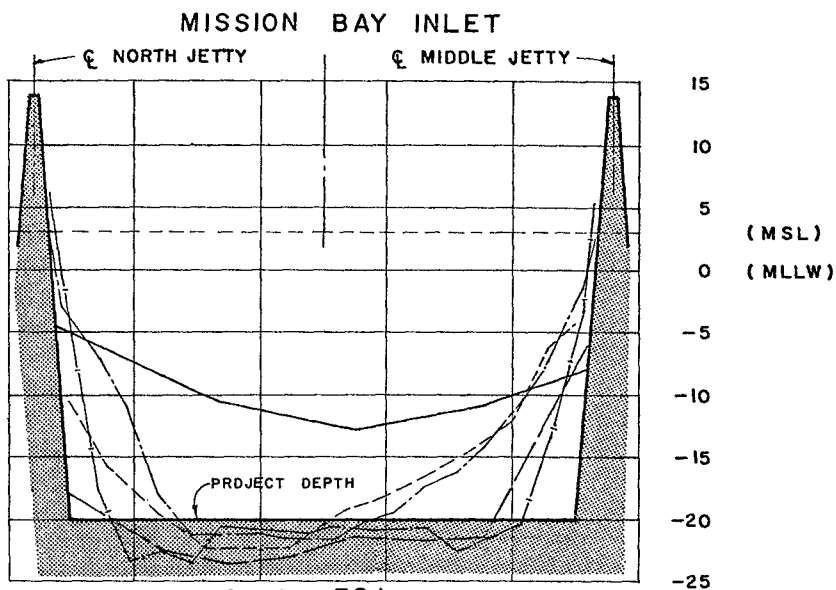
BEFORE CONSTRUCTION
GENERAL CONFIGURATION OF BAY AT HIGH TIDE
1947 NO. I

START INTERIOR CONSTRUCTION
NOTE CONFIGURATION OF NATURAL INLET
FEB. 1948 **NO. 2**

START ENTRANCE CONSTRUCTION
MIDDLE AND SOUTH JETTIES
JULY, 1948   NO. 3

COMPLETION OF FLOOD **CONTROL CHANNEL** OUTLET
MIDDLE AND **SOUTH JETTIES**
FEB. 1949   **NO. 4**

NAVIGATION AND FLOOD CONTROL CHANNEL JETTIES
COMPLETION OF NORTH JETTY
NOV. 1950   **NO. 5**

NAVIGATION AND FLOOD CONTROL CHANNELS
MIDDLE JETTY CLOSED, CREATING **TWO CHANNELS**
SEPT. 1951   **NO. 6**

DEVELOPMENT OF WEST MISSION BAY
NOTE CLOSURE OF FLOOD CHANNEL
JAN. 1953   **NO. 7**

START OF DEVELOPMENT OF EAST MISSION BAY
NOTE COMPLETED DREDGING OF ENTRANCE CHANNEL
JAN. 1958   **NO. 8**

COMPLETION OF MISSION BAY - SAN DIEGO RIVER PROJECT
1963   NO. 9

HYDRAULIC MODEL STUDY - MISSION BAY ENTRANCE
1969   NO. 10

CONSTRUCTION PROCEDURE

In any project of this magnitude, construction problems during an extended construction period (1946 to 1963) can create problems and misunderstandings. The time s quence for creation of the bay and the jettied entrance channel was as follows:

1946: Dredging and filling of interior portions of the bay were initiated by the City. (Photo 2, 1948).
May 1948: The Corps of Engineers initiated construction of the south (flood control) jetty and the middle (combined flood control and navigation) jetty. (Photo 3, 1948); Photo 4, 1949).
June 1949: The Corps of Engineers initiated construction of the north (navigation) jetty.
November 1949: The south and middle jetties were completed.
September 1950: The north jetty was completed (Photo 5, 1950).
June 1950: A pilot channel (300 feet wide) was dredged to the -8 foot MLLW depth to divert tidal flow of the bay to between the navigation jetties. (Photo 5, 1950).
February 1951: The flood control channel was completed and the final section of the middle jetty was closed, separating the Flood Control Channel from Mission Bay. (Photo 6, 1951).
March 1951 to Nov. 1954: The Corps of Engineers project was shut down because of the Korean War. During this time period, the City essentially completed their dredge and fill program for the bay westerly of Ingraham Street (Photo 7, 1953).
December 1954: Dredging of the outer entrance channel by the Corps of Engineers was resumed.
May 1955: An experimental attempt was made to seal the jetties to prevent littoral sand from entering the navigation channel through voids in the jetty cap rock.
July 1955: The outer entrance channel was completed to full width and depth.
January 1957 to December 1957: The Corps of Engineers dredged a portion of the main channel and Quivera Basin to a depth of -20 feet MLLW. This relatively coarse sand was pumped to the eastern perimeter of the bay to stabilize the mud deposits along U. S. 101 Highway. This dredging essentially permitted full and unimpeded tidal flow through the West Bay, the main channel and the entrance channel. (Photo 8, 1958).
July 1958: The Corps of Engineers dredged the main channel and the west anchorage basin (Mariner's Basin) to a depth of -15 feet, MLLW. At the same time, the City initiated a 12 million cubic yard dredging contract to complete the development of the east bay. This project, completed in 1963, essentially completed the City's dredge and fill program, and while the natural tidal prism was reduced by about 10%, flow characteristics were greatly improved by elimination of the large marshy areas with flat slopes between high and low tide. (Photo 9, 1963).
August 1959: The middle and north jetties were sealed with concrete grout from mean lower low water to the +4 foot elevation through the littoral zone to prevent future shoaling of the navigation channel by intrusion of littoral sand through the large voids in the cap rock.

<u>October 1966 to June 1969</u>: Field studies, resulting in a hydraulic
model study were made by the Corps of Engineers to develop a solution
for occasional undesirable wave action in the two deepwater
anchorage basins (Photo 10, Hydraulic Model Study).
<u>1972</u>: Maintenance dredging is planned to remove about 300,000 cubic
yards of sand from the entrance channel.

DISCUSSION OF PROBLEMS ENCOUNTERED
DURING CONSTRUCTION

The shutdown of this project as a result of the Korean War had
a most adverse effect. While the flood channel was completed, only
a minimum effort was made to open the bay to the sea by dredging a
pilot channel through the barrier beach. The pilot channel was to
be 300 feet wide to the -8 foot depth (Fig. 2a). In other words,
looking at O'Brien's equilibrium equation the cross-sectional area
of less than 4000 feet was completely inadequate and the channel
shoaled and spread the full width between jetties to form a very
dangerous bar. (Figs. 2c and 2d). From an engineering point of view
this was a very successful maneuver because it is in accord with the
equilibrium relationship and because of the west coast tidal
character of getting maximum tidal run from MHHW to MLLW, the minimal
cross-sectional area of the entrance channel increased to about
11,000 ft.^{2} by July 1954, and over 250,000 cubic yards of sand were
scoured from the channel and apparently carried out to sea.
However, the boatmen were not adequately informed as to the hazards
of this partial channel, and before its closure by City Ordinance
in 1953, several boats were capsized and 11 lives lost.

Another major problem was the intrustion of littoral sand into
the entrance channel. However, until 1954, in Southern California,
it was the practice to only bring core rock, or "C" rock up to MLLW
for jetties and then to continue construction through the wave zone
with large cap rock. Thus both navigation jetties at Mission Bay
consisted of 2 to 8 ton cap stone from MLLW to the +14.0 foot
elevation. It was realized during the initial dredging of the
entrance channel in early 1955 that a very appreciable amount of
littoral sand was being carried through the voids of the cap rock
and shoaling the navigation channel. In May 1955 a plan was conceived
to introduce a concrete grout diaphram into the center of the cap
rock section (7). After considerable testing, this was accomplished
by drilling holes along the centerline of the structure and forcing
a mixture of concrete and bentonite (driller's mud) through the holes
and into the central voids of the cap rock. The intent was to create
a diaphram with a minimum elevation of +4 feet. Some 2200 feet of
the middle jetty and 600 feet of the north jetty were sealed in this
manner.

During this same period of developing the entrance channel, it
was observed that, all too frequently, waves were either breaking in
the entrance channel or were so steep as to constitute a serious
hazard to small boats. As a final move, in June 1960 the outer 1000
feet were dredged to the -25 foot depth. This appears to have greatly
improved the navigability of this outer portion.

    The remaining inlet problem at Mission Bay is excessive wave
action inside the bay. It has been learned at Mission Bay, Alamitos
Bay, Newport Harbor and Marina del Rey that while they are excellent
harbors, so far as navigability and low maintenance are concerned,
these excessively wide entrances do admit a great deal of wave
energy which must be disposed of. Alamitos Bay is partially
sheltered by the outer harbor breakwater and much of the interior
perimeter is still in sand beaches which can absorb considerable
wave energy. Much the same is true at Newport; the jetties are
skewed some 20° to the south,and the north jetty acts, in part, as
a breakwater. Newport Bay also consists mostly of sand beaches
around its perimeter, and has had no interior wave problems. A very
bitter lesson was learned at Marina del Rey, where subsequent to
construction of jettied entrance, the interior walls were changed
from rock revetment to vertical concrete bulkheads which almost
totally reflected wave energy. An offshore breakwater had to be
constructed across the entrance to keep this wave energy out of
the boat basins. At Mission Bay, a wave problem exists in the two
deepwater anchorages: Quivera Basin and Mariner's Basin. Very
soon after installation of boat slips in 1959 and 1960 in Quivera
Basin, there were complaints of waves as high as 2.5 feet at the
berths with damage to both boats and floats. Studies were made in
the field (8) and by hydraulic model at the Corps of Engineers'
Waterways Experiment Station (9) and it was concluded that the main
problem was excessive wave energy reaching the inshore end of the
entrance channel. The inshore end of the entrance channel makes
a 90° left into the main channel, and the shore is protected by a
semi-circular rock revetted slope. This revetment not only tends
to reflect too much wave energy, it also tends to focus this
reflected energy towards the entrance to Mariner's Basin. Several
corrective measures were tested and the most promising was to
convert the revetted semi-circular end of the channel to a series
of straight revetted sections in echelon that would tend to reflect
this energy back out the entrance channel to sea (Photo 10).
Estimated cost of this modification is about $1,000,000, and until
there is further development of the two basins and the need is
strongly demonstrated, no action is planned.

REVIEW OF DESIGN 1972

    The design of the inlet for Mission Bay, as developed in 1946-
1949, has since proved to be fundamentally correct. Dean O'Brien
reviewed his work in 1969 (10) and arrived at the basic equation:
$A = 4.69 \times 10^{-4} P^{0.85}$, expressing P in cubic feet rather than his
original expression of square-mile feet. Thus the minimal cross-
sectional area of the entrance channel for equilibrium flow would
still be in the order of 9000 square feet. Since completion of the
entrance channel dredging in 1959, the cross-sectional area has
varied from 16,400 sq. ft. to the design area of 19,800 sq.ft. which
is well above the minimal. The first maintenance dredging of the
entrance is scheduled for the latter part of 1972. Making due
allowance for sand entering the channel before completion of jetty
sealing, the average maintenance dredging of the entrance between
1954 and 1972 amounts to less than 20,000 cubic yards per year.

In Feb. ,1972, Professor J. W. Johnson published a paper comparing the various tidal inlets of California, Oregon and Washington (11). His work confirmed the equilibrium curve developed by O'Brien. He realized, as did O'Brien, that factors other than tidal prism should have an influence on the equilibrium cross-sectional area of the inlet. In an attempt to prove this point, he compared the characteristics which he believed should have certain modifying effects on the equilibrium ratio. He gave identifying symbols to the plotting points for inlets with two,one or no jetties and with various degrees of exposure to wave action. He also made separate plots of his data adding such parameters as tide range and tide-cycle duration. Surprisingly, these factors and parameters showed no sifnigicant modifying trends. All inlets plotted within reasonable proximity of the equilibrium curve with the exception of Morro Bay, Marina del Rey, Alamitos Bay and Newport Bay, all of which showed entrance sections far larger than their tidal prisms should have allowed. Prof. Johnson stated that these radical departures from the norm appeared to be "primarily a function of wave climate and the limited movement of littoral drift as affected by long jetties and the possible initial over-dredging of the entrance channels". He concluded that since these inlets and a few others, including the Mission Bay Entrance,showed no evidence of shoaling, they were not in equilibrium in the usual sense for unimproved inlets because of the effects of limited wave action, low littoral drift or long jetties. It is apparent from Figure 3 of this study that, at least in the case of San Diego Bay, Mission Bay, Newport Bay, Marina del Rey and Alamitos Bay, the inlet has not been over-dredged by mistake, but the cross-section has been deliberately increased beyond "equilibrium" limits. This was done to reduce the tidal currents and to prevent appreciable sand transport in the entrance channel.

The various relationships between the tidal prism and the area of the inlet since start of construction in 1946 are shown in Figure 3 and Table 1. lt is apparent that there has been a very major change in the relationship of tidal prism to inlet area with a resultant reduction in tidal velocities, yet no significant entrance shoaling has occurred as the equilibrium formula would have predicted.

During the original design of Mission Bay and its inlet, it was only possible to make a rough approximation of tidal current velocities through the critical sections. Since 1948, a great deal of study has been done on this subject and one of the most widely accepted methods for computing velocity of inlet currents was developed by Dr. G.H. Keulegan, as described in the U.S. Army Corps of Engineers' Committee on Tidal Hydraulics, Technical Bulletin No.14. His procedure takes into account the tidal oscillation in the ocean, the physical properties of the inlet channel, the physical properties of the basin and inflow from streams or rainfall, and it develops a mathematical relationship between the tidal range in the basin and the maximum velocity in the connecting channel.

TABLE 1
MISSION BAY INLET STUDY
RELATION TIDAL PRISM TO INLET

Date	Min.CrossSection-Area of Inlet A(Sq.Ft)		Tidal Prism Spring Tide Range P (Cu.Ft).		Ratio P/Ax10^4	
1947	7.2×10^3	(1)	4.5×10^8		6.3	
July 1954	1.11×10^4	(2)	4.4×10^8		4.0	
Aug. 1955	1.95×10^4	(3)	4.4×10^8		2.1	
July 1958	1.55×10^4		4.3×10^8		2.8	
Jan. 1959	1.86×10^4		4.3×10^8		2.3	
June 1963	1.78×10^4		4.0×10^8	(4)	2.3	
May 1970	1.64×10^4		4.0×10^8		2.4	
June 1971	1.72×10^4		4.0×10^8		2.0	(6)
Fall 1972	1.98×10^4	(5)	4.0×10^8		2.0	(6)

1. Natural Channel, including San Diego River
2. Adjusted pilot channel, dredged in June 1950
3. Entrance channel dredged to full dimensions
4. Tidal Prism completed to full dimension
5. Proposed Maintenance dredging of entrance channel
6. The O'Brien equilibrium equation for the relationship between the entrance channel with two jetties and the tidal prism for a tidal prism(P) of 4.0×10^8 cu.ft. gives an equilibrium cross-sectional area(A) of 1.0×10^4sq.ft. and P/A is 4.0×10^4. This is twice the value of P/A for a "Non-Scouring" channel.

Mr. John M. Nichol, Moffatt & Nichol, Engineers, had occasion to use Dr. Keulegan's method in a study of currents at Channel Islands Harbor (13) in 1970. He developed the following maximum entrance current velocities for the "non-scouring" entrance channels of Southern California:

San Diego	2.1 ft/sec.
Mission Bay	1.9
Newport Bay	1.5
Alamitos Bay	1.1
Marina del Rey	0.4

Thus it is seen that "tidal scour" is a very minor or non-existant factor in design and maintenance of these particular harbors.
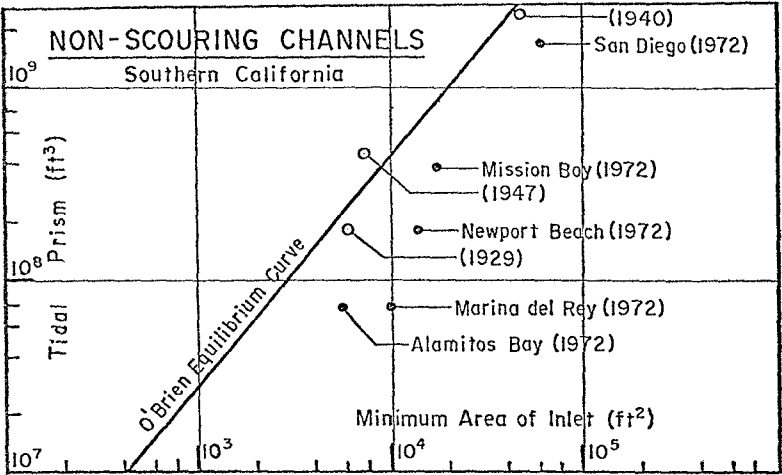
FIGURE 3
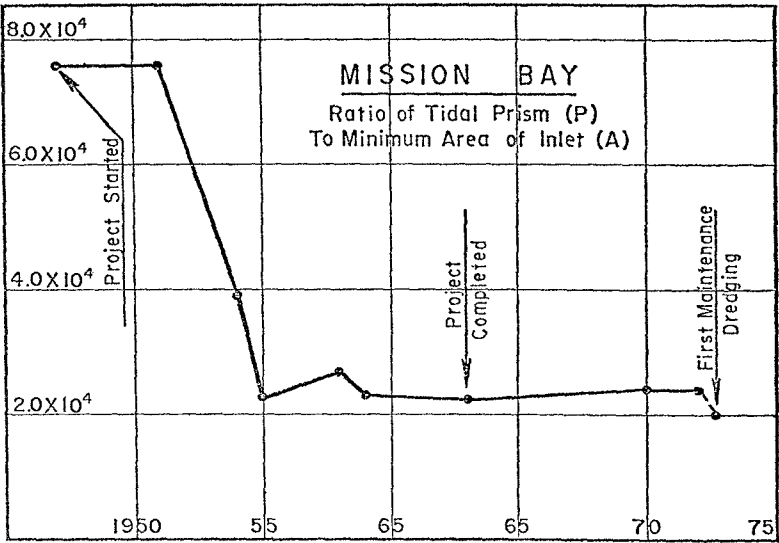
FIGURE 4

CONCLUSIONS

The principle of a "non-scouring-non-shoaling" channel, such as was developed for Mission Bay in 1946-49, is sound so long as the following criteria are satisfied: 1) there must be no exterior supply of shoaling material either from upland drainage or from littoral drift; 2) tidal velocities must be reduced to where there is no movement of bottom sediments within the bay or the entrance channel; 3) the entrance channel must be of sufficient depth to prevent appreciable movement of bottom sediments by wave induced currents.

Problems created by this type channel are: 1) possibility of such low tidal velocities as to fail to create adequate flushing action and cause pollution problems; 2) a wide entrance channel may permit excessive wave energy to be delivered to the inner basins. Measures must be taken to properly dispose of this energy by either absorption or reflection.

The most serious problem at Mission Bay resulted from the suspension of work on the project from 1951 to 1955 because of the Korean War, and the failure of the boating public to appreciate the hazards of using a partially dredged entrance channel. Completion of the entrance channel to full dimension eliminated this hazard.

A jetty-design deficiency had to be corrected because of the use of cap rock only in the tide and wave zone above mean lower low water level. The large voids (35 to 40%) inherent in caprock placement permitted an excessive amount of littoral sand to pass through the jetty into the channel. Sealing of portions of the jetties to the +4 foot elevation effectively reduced this shoaling factor.

Since completion of the entrance channel to full design dimension in 1959, there has been no serious shoaling or navigation problems. The first maintenance dredging scheduled for the latter part of 1972 indicates an average annual rate of shoaling of less than 20,000 cubic yards.

* * * *

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