CHAPTER 49

JETTY FOUNDATIONS ON FINE SEDIMENTS

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SYNOPSIS

The Secretaría de Marina Nacional of Mexico, contemplates the construction of two long jetties at the mouth of the Grijalva River to permit safe navigation into the port of Frontera in the state of Tabasco, Fig 1. The port of Frontera is located in the estuary of the Grijalva River 9Km. from its mouth. The proposed jetties should reach into the sea to a depth of water of 6 mts. This requires a length from the mouth of the river of about 2000 mts, Fig 2. Rock fill jetties constructed in the past in this area on the fine sediments have failed by spreading and penetration into the fine co hesionless sediments encountered at the sea bottom. Heavy structures cannot be constructed on account of the low shearing strength of the submarine delt clay deposits that may be encountered at the mouth of the Grijalva River.

Subsoil investigations were performed by the author to learn the mechanical properties of the materials at the mouth of the Grijalva River, and made possible the design of light-weight and strong structural jetties to re sist the sea and river forces to which these structures will be subjected. The problem involved and stability considerations of the jetties are explair ed by the author in this paper.

GEOTECHNICAL STUDIES

Subsoil investigations at the mouth of the Grijalva River indicated a surface deposit of about 5 mts thick of fine sand with frequent diameter ra ging between 0.15 and 0.20 mm. This material is the product of the present sedimentation of the river and covers uniformly all the mouth and extends i the sea at least 3 Km. from the mouth of the river. The typical subsoil pr file obtained from one of the continuous 4 inches undisturbed sample cores ken up to a depth of 37 mts is shown in Fig 2. The thickness of the upper sand deposit at the river mouth changes because of deposition and erosion s by the currents of the river during the different seasons of the year, Fig .

Overlain by the fine sand deposits it may be encountered a stratificat of gray clayey-silt with small shells and average thickness of 1.2 mts. Th water content of this material assumes a value of 60-70%. Consolidated undrained tests in this material give 0.17 Kg/cm^2 for cohesion and 22°50' for the ϕ_{cq} angle of internal friction. This stratification containing small ells is a good marker to correlate the stratigraphy from one bore hole to a

Fig. 1. Picture of mouth of Grijalva River.

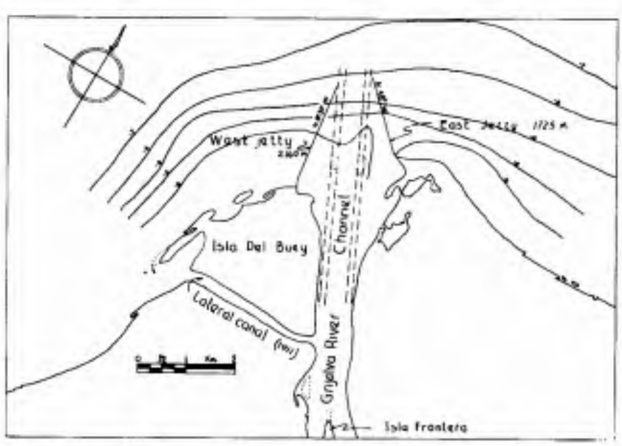
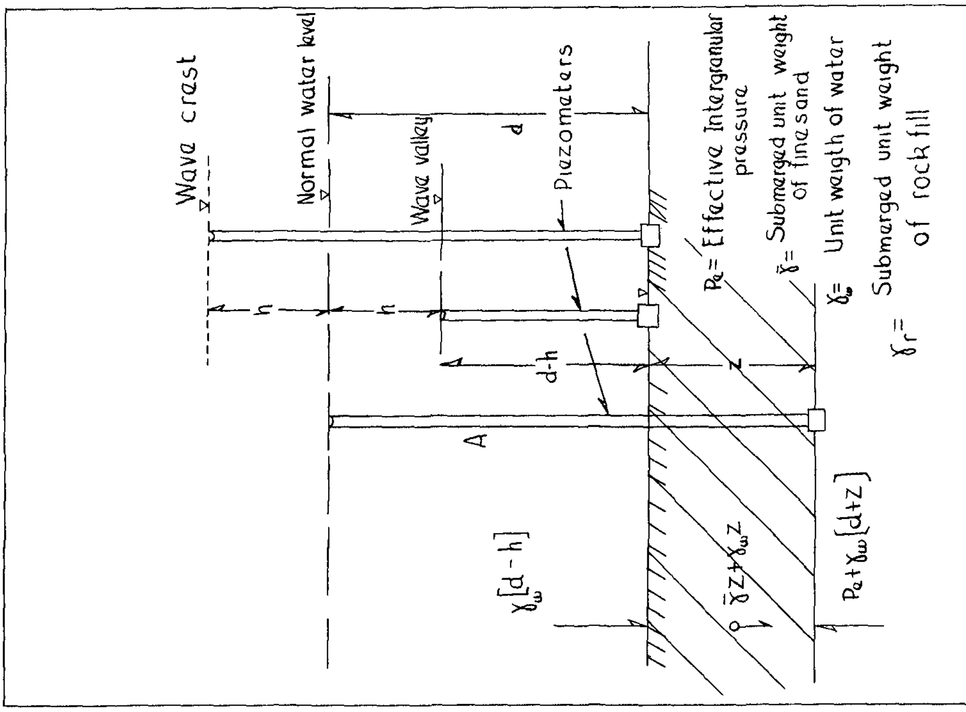
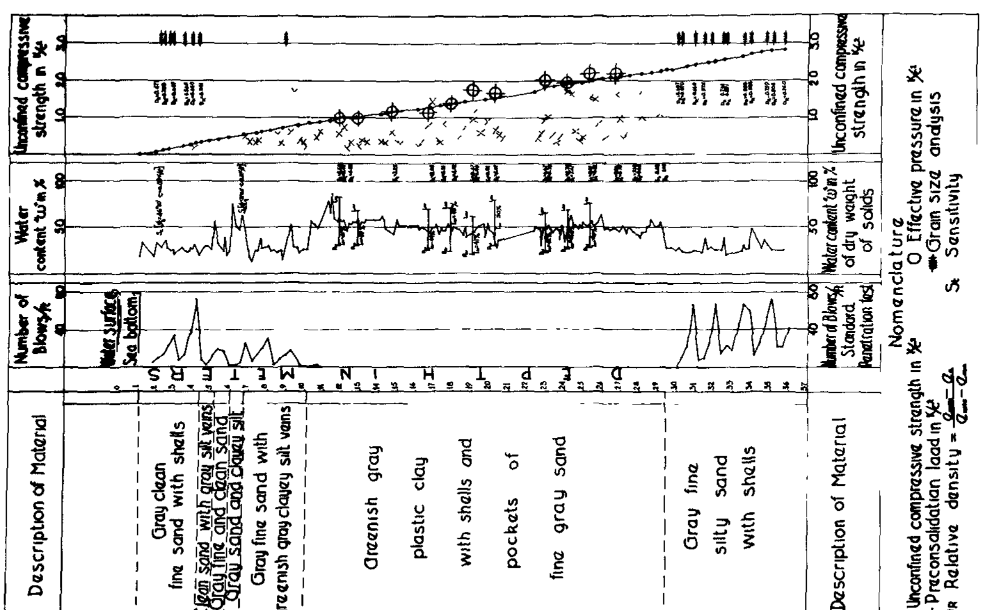


Fig. 2. Proposed location of jetties.


Wave crest
Normal water level
Wave valley
Piezometers
d
h
h
d-h
A
$\gamma_w [d-h]$
$P_e + \bar{\gamma} Z + \gamma_w Z$
$P_e + \gamma_w [d+Z]$
Z
P_e = Effective Intergranular pressure
$\bar{\gamma}$ = Submerged unit weight of fine sand
γ_w = Unit weigth of water
γ_r = Submerged unit weight of rock fill


Description of Material
Number of Blows/ft
Water content 'w' in %
Unconfined compressive strength in T/ft²
Water surface
Sea bottom
Gray clean fine sand with shells
Clean sand with gray silt veins
Gray fine and clean sand
Gray sand and clayey silt
Gray fine sand with greenish gray clayey silt veins
Greenish gray plastic clay with shells and pockets of fine gray sand
Gray fine silty sand with shells
Number of Blows/ft Standard Penetration test
Water content 'w' in % of dry weight of solids
Nomenclature
X Unconfined compressive strength in T/ft²
Preconsolidation load in T/ft²
O Effective pressure in T/ft²
Grain size analysis
St Sensitivity

other. Following this stratum a uniform deposit of silty sand with little clay may be found with a thickness of 3.1 mts correlating very well in all the bore holes. The average water content is 23% and the unconfined compressive strength as low as 0.2 Kg/cm^2.

From 10 to 29.5 mts depth it may be encountered a thick greenish-gray clay deposit with microscopic shells, and underlain by gray silty sand with small shells. This deposit is the product of the sedimentation of the finer alluvial sediments deposited in the sea. This fines coming into the sea flocculated and settled forming the submarine delta of the river. However, the very fine submarine sediments found in the present at the mouth of the river were undoubtedly formed when the mouth was several kilometers behind its actual position. The recent advance of the mouth of the Grijalva River toward the sea has produced the upper coarser sediments underlain by the clay deposit.

The average water content of the clay deposit is about 55% and shows practically constant with depth Fig 3. The clay is normally consolidated. This fact may be demonstrated by the good agreement shown between the break in the compressibility curves and the computed overburden pressures.

The shearing strength of the clay deposit between 10 and 29.5 mts depth obtained from unconfined compression tests shows a very erratic value, probably because of the strong difference in the salinity of the water during the process of sedimentation and flocculation of the fine grains. This fact may have had an important influence in the shearing strength of the clay; However, the minimum value of the shearing strength is of 0.15 Kg/cm^2 at 10 mts depth and increases only to 0.2 Kg/cm^2 at 29.5 depth.

From numerous consolidation tests on laterally confined specimens it was found an average value of m_v = 0.033 to 0.038 cm^2/Kg based on a period of 100 years, when the secondary settlement is taken into account (1). Primary consolidation takes place in only 20 years after load application.

Using the settlement analysis (1)(2) equations:

$$S = m_{v_1} F(T_v)\, \Delta p \cdot H \quad ; \qquad t < t_a$$

$$S = \left[m_{v_1} F(T_v) + m_t \log \frac{t}{t_a} \right] \Delta p \cdot H \quad ; \quad t > t_a$$

The corresponding values of the parameters are:

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(1) Ecuación Completa de Consolidación para Depósitos de Arcilla que Exhiben Fuerte Compresión Secundaría, by Leonardo Zeevaert, June 1957 Published in Revista Ingenieria.

(2) Consolidation of Mexico City Volcanic Clay by Dr. Leonardo Zeevaert, Proceedings Joint Meeting of ASTM and SMMS, Mexico City , Dec.9-13-57

$$m_{v_1} = 0\,027 \quad \text{to} \quad 0\,035 \quad cm^2/kg$$

$$m_t = 0.011 \quad \text{to} \quad 0.012 \quad cm^2/kg$$

$$T_{va} = 1.0$$

$$C_v = 0\,00159 \quad \text{to} \quad 0.00146 \quad cm^2/sec$$

$$T_v = \frac{4C_v}{H^2} \cdot t \quad ; \quad t_a = \frac{T_{va}}{4C_v} \cdot H^2$$

$$t_a \doteq 20 \text{ years.}$$

The total thickness of the deposit may be taken as 19.5 mts drained ε top and bottom.

SUBSIDENCE PROBLEM

When a heavy rock fill is constructed on these fine sand, silt and c sediments to form a jetty or any water wave protection, it is necessary t consider in the design the following phenomena:

a.- penetration of the rock into the fine cohesionless sediments because of the possibility of spontaneous liquefaction.

b.- spreading of the slopes of the rock fill because of material erc sion in the foundation.

c.- subsidence of the fill because excessive shearing stresses in th underlying clay deposit.

d.- large settlement of the fill because of excessive compression.

Items (a) and (b) are produced by wave action and (c) and (d) becau of exseeding, respectively, the allowable mechanical properties of shear strength and compressibility of the soft clay deposit.

(a) Penetration of the rock into the fine cohesionless material is used by the reduction of shearing strength in the sand at the passage of water waves. That is to say, as a water wave passes over certain point the slope of the jetty the hydrostatic pressures in the soil mass cannot just themselves to the instantaneous hydrostatic excess water pressures. erefore, an important hydrodynamic lag may be created in the sand suppor the rock material of the jetty, thus the hydrostatic excess pressure cre momentarily an spontaneous liquefaction condition of the sand permitting rock to penetrate into the fine sand. This effect is a function of the wave height. To demonstrate this phenomenon let us consider Fig 4 a wave coming along the jetty. The crest rises from the normal water level in approximately "h" and as the valley of the wave passes along, the water level drops from the average level in approximately "h". One piezometer A installed at depth "z" into the very fine sand deposit will preserve the normal or average water level as the water wave passes rapidely. Therefore, an important water uplift pore pressure in the soil will take place as the valley of the wave passes over the point considered. Taking in consideration, Fig 4, the forces acting for equilibrium in the upper part of the deposit of thickness "z" may be obtained the following equilibrium equation:

$$p_e + \gamma_w (d+z) = \bar{\gamma} z + \gamma_w z + \gamma_w (d-h) \quad \text{------} \quad (1)$$

thus:

$$p_e = \bar{\gamma} z - \gamma_w h \quad \text{------} \quad (2)$$

The effective intergranular pressure at depth "z" is equal to the weight $\bar{\gamma} z$ of the submerged sand, minus the uplift pressure $\gamma_w \cdot h$ produced by the wave height. The equilibrium is unstable up to a depth where: $p_e = 0$

hence; $z = \frac{\gamma_w}{\bar{\gamma}} \cdot h$, since: $\frac{\gamma_w}{\bar{\gamma}} \doteq 1$, then $z \doteq h$

This implies that the waves produce an unstable condition in the cohesionless fine sand and silty sediments of the bottom of the sea to a depth approximately equal to the semi-height of the waves.

A heavy rock fill will penetrate into the fine sediments to a depth where p_e starts to be larger than zero.

Therefore, if the submerged weight of the rock fill is $\bar{\gamma}_r$ and its thickness above the sand is D then:

$$\bar{\gamma}_z - \gamma_w h + \bar{\gamma}_r D = 0 \quad \text{------} \quad (3)$$

from which:

$$z = \frac{\bar{\gamma}_r \cdot D - \gamma_w h}{\bar{\gamma}} \quad \text{------} \quad (4)$$

The value of "z" represents the theoretical depth at which the rock fill will penetrate into the sand. Practically no penetration will take place for depths of fill:

$$D \geq \frac{\gamma_w}{\bar{\gamma}_r} \cdot h \qquad \text{------} (5)$$

However, at the foot of the slope there is always the tendency to have a penetration equal to that given by formula (2).

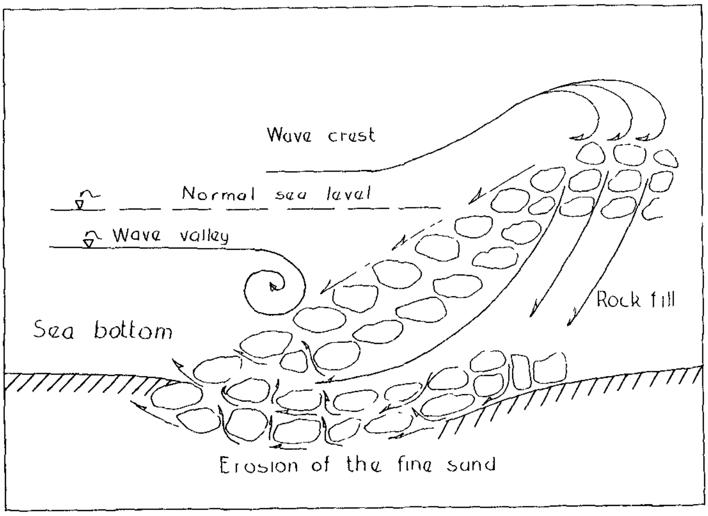

Fig. 5. Erosion in rock fill.

(b) The spreading of the rock fill because of erosion is a very impor tant phenomenon that may take place because of erosion at the foot of the slope of partially submerged rock fills on fine cohesionless sediments, lil fine sand. This phenomenon is facilitated as the fine sand becomes loose during spontaneous liquefaction as explained above (a). Let Fig 5 be the slope of a partially submerged rock fill, as the crest of the water wave co mes along the slope, and the water fills up the voids left in the rock fil: Following the valley of the wave, because of the lower water level, the wa ter flows strongly out from the inside of the rock fill, producing a stron erosion in the fine sand at the base, Fig 5. Therefore, the stability of the slope may be lost and spreading may take place. Furthermore, the com- bined effect of uplift pressure, as explained before, and the horizontal f ce produced by the rapid drawdown as the valley of the wave passes along m produce eventually a total spreading of the fill. The shearing strength a the base of the rock fill may be not enough to counteract the internal wat force developed in the rock fill. A spreading failure of this type in con junction with penetration in the fine sand was originated in old jetties c structed in this location in 1911. The observations made in 1950 to find the section of this rock jetties showed they had spread and penetrated str ly into the sand, demonstrating this type of failure.

(c) Breaking of the rock fill into the ground may be controlled by not exceeding the shearing stresses in the soft clay deposit.

(d) Excessive settlement may be avoided by not overloading the clay deposit. Since the clay deposit is not of high compressibility it appears from settlement analysis that the computed settlements with safe bearing loading capacity will produce reasonable settlements that can be taken safely by the structure.

STABILITY OF PROPOSED SECTION

The decision of the proposed section was obtained after carefull study of the subsoil mechanical properties in conjunction with the theoretical studies concerning the possibility of spontaneous liquefaction of the fine sand sediments, erosion and the phenomenon of possible spreading of the partially submerged rock fill. Furthermore, since the shearing strength of the clay is only 0.15 Kg/cm^2 it was decided to abandon, for safety, the idea of a partially submerged rock fill jetty. After studying other possible sections the author decided to recommend a structure of reinforced concrete boxes built with a system of sheet piles driven previously and forming a cofferdam-like structure, Fig 6. The structure, thus formed is confined laterally against strong erosion of the fine sediments by means of a minimum rock fill placed at the bottom of the sea against the proposed structure. With the purpose of resisting the large dynamic lateral forces because of the impact of the breaking waves against the structure, it was necessary to anchor properly the sheet-piles into the clay deposit. In this fashion it was possible to make a very favorable use of the shearing strength of the clay.

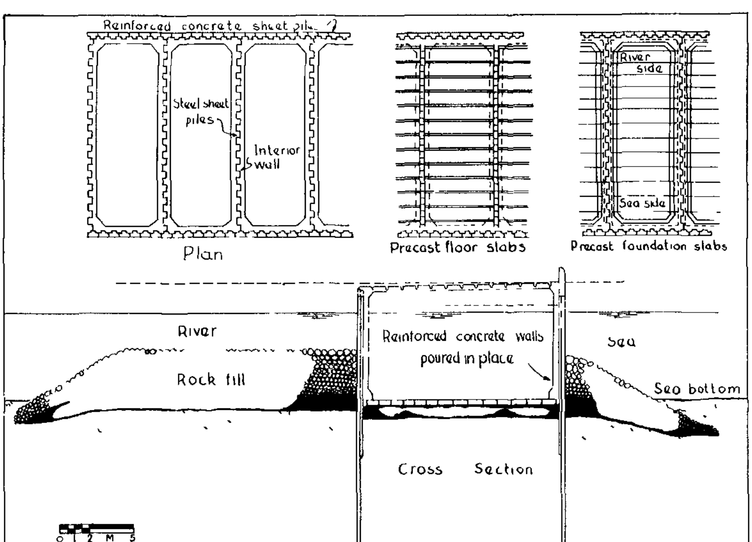

Fig. 6. Proposed section.

However, the coffer-dam formed of hollow reinforced concrete boxes Fig 6, is alone not enough to hold the large thrust of the breaking waves on the structure. Therefore, to increase the safe bearing capacity it was necessary to confine the bottom of the sea close to the reinforced concret structure by means of a rock fill, thus increasing also the stability of tl structure as a whole and reducing the dimensions and cost of the central pc tion.

On the river side a larger rock fill is provided to avoid any possibl sliding shear failure either along the clay or through the fine sand. The cross section of the proposed jetty for 6 mts depth is shown in Fig 6. Th central structural part of the jetty shows perfectly fixed by the sheet-pi to the subsoil.

The transverse walls formed by sheet-piles of steel "Z" section are tl basic elements of strength in the jetty. The depth and spacing of these diaphragms was computed in order not to exceed the allowable shearing stre of the clay during the large transient forces applied by the waves on the side of the jetty. The dynamic force is transmitted to the diaphragm and transmitted by shear into the subsoil. The reinforced concrete, Fig 6 box type structure above the bottom of the sea makes the entire structure to w as a unit.

The rock fill used to confine the structure was computed following co clusions on the spontaneous liquefaction and erosion as described before a taking into account that on a long term basis the rock fill at the foot of the slope will penetrate one-half the wave height into the fine sand, because the dynamic action of the waves, Fig 6.

Using the properties of compressibility obtained from consolidation t as reported in the first part of this paper, it was found that the settlem of the jetty would be on the order of 17.4 cm in 100 years. From which 15 cm. will be primary consolidation taking place in approximately 20 years. The compressibility of the clay deposit is rather uniform in a large exten therefore differential settlements will be of minor importance to the beha and maintenance of the jetties.

Pre-stressed concrete was recommended by the author for the construct of all the pre-cast reinforced concrete elements as the lateral sheet-pile foundation slab and floor slab. The walls will be poured in place. The f slab is designed to hold truck loads up to H-15.

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