SEISMIC RESPONSE OF PILES IN FINE SAND

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SUMMARY

The piers of the Navy Yard in the State of Veracruz, Mexico, located at the Coatzacoalcos River Bank suffered large permanent displacements during an earthquake 6.5 magnitud with epicenter at about 35 Km from the site. The author describes in the paper a tentative quantitative correlation of the phenomenon using the geometry of one of the pier units and based on the subsoil conditions available.

The problem is analysed theoretically from a practical engineering point of view establishing the pile-soil interaction, and considering that during the strong ground motion high pore water pressures developed in the sand deposit, thus reducing the lateral rigidity of the sand supporting the piles of the pier. The results of the theoretical calculations obtained by the method proposed by the author are confronted with the damage observations at the site.

INTRODUCTION

In August 26, 1959 a mayor earthquake was recorded with epicenter in the Gulf of Mexico with latitude 18°27'N, longitud 94°16'W and approximately at 35 Km from the mouth of the Coatzacoalcos River in the State of Veracruz, Mexico. The earthquake was rated 6.5 Richter Magnitud and its intensity estimated on the order of VII M.M. with maximum ground surface acceleration of 200 gal (Ref. 1). The installations at the Navy Yard located at the river banks close to the mouth of the river, suffered severe damage. Observations by the author just after the earthquake reported permanent relative displacements on the order of 25 cm between pier units supported on steel pipe piles, Fig 1 and 2. The pipes used are 20 cm diamter standard steel pipes driven through the loose sand to point bearing on a soft sand stone. The geometry of the pier analysed, the mechanical properties of the pipe piles and the subsoil characteristics are shown reported in Fig 3.

The phenomenon was analysed considering that during the seismic motion the maximum ground surface acceleration reached 200 gal at the dredge line. The following actions are assumed to have taken place:

1) High pore water pressures in the sand deposit reducing the soil rigidity during the seismic motion.

2) Amplification of the ground surface acceleration at the pier deck elevation.

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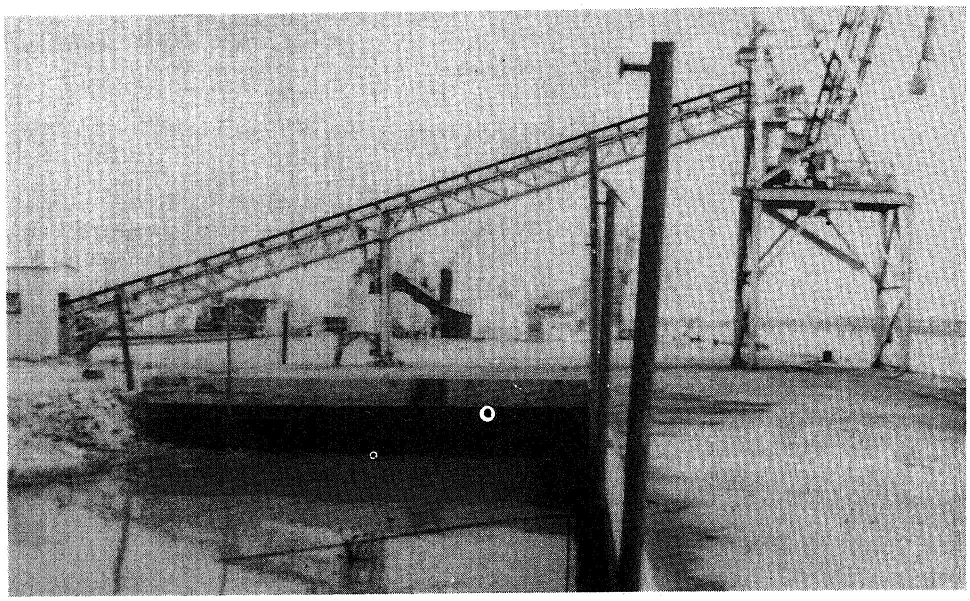


FIG 1. PIER AT THE COATZACOALCOS RIVER

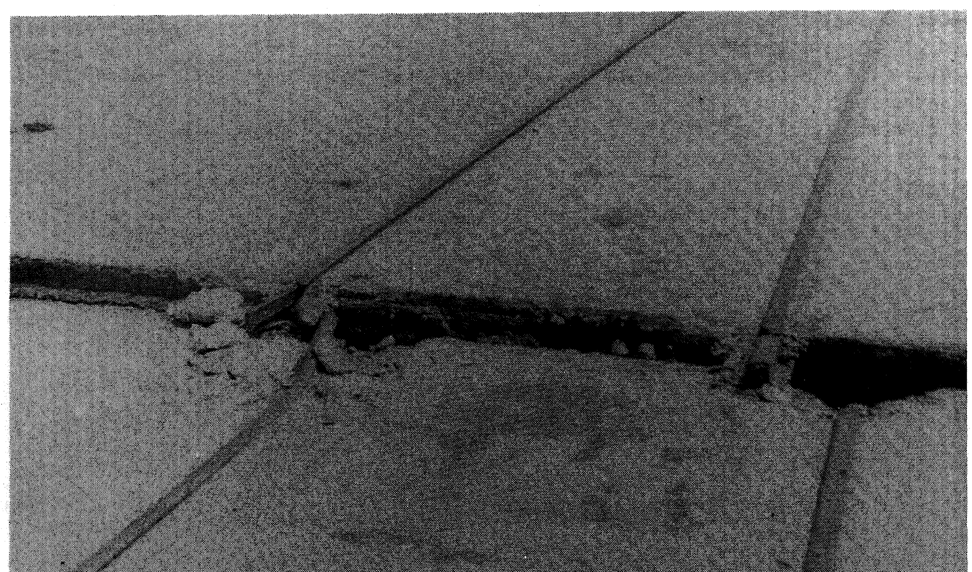
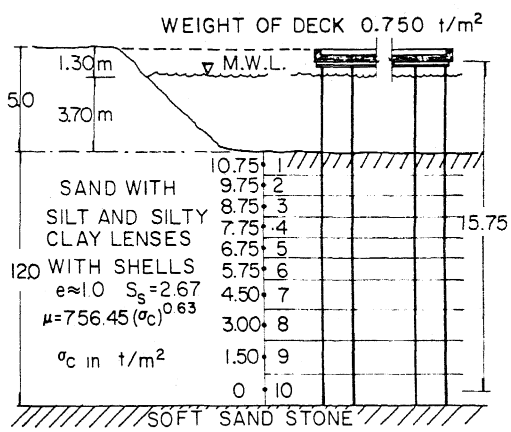


FIG 2. DISPLACEMENT BETWEEN PIER UNITS

The evaluation of points (1) and (2) permitted the computation of the approximate behavior of the pier to be compared with the observed damage.



1).- DECK ON 8"ϕ STANDARD STEEL PIPES 1.5m CENTERS PROPERTIES: $I = 1118.83\ cm^4$, $A = 3.48\ cm^2$, $I/c = 110.72\ cm^3$ $EI = 240.55\ t\cdot m^2$, CHELLIS, D.R, PILE FOUNDATIONS (SECOND EDITION) p.585, Mc. GRAW-HILL. (1000 KILOGRAMS = 1 ton)

2).- WEIGHT OF DECK PER PILE: 1.69 ton.

FIG 3. CROSS SECTION OF PIER

SUBSOIL BEHAVIOR

To estimate the subsoil behavior it was necessary to learn on the soil rigidity. The author performed in the past dynamic soil investigations for a similar fine sand at the mouth of the Grijalva River in the Gulf of Mexico, located in the State of Tabasco, (Ref. 3). The results of this investigation yielded the following value for the loose fine sand dynamic soil rigidity

$$\mu = 756.45(\sigma_c)^{0.63} \text{ton/m}^2 \qquad (1)$$

in which σ_c given in ton/m² is the confining effective stress at the depth where μ is required. Hence, knowing the maximum seismic pore water pressure U_{sis}, the seismic sand rigidity during the earthquake may be estimated by

$$\mu_{sis} = 756.45(\sigma_c - U_{sis})^{0.63} \qquad (2)$$

The next problem was to determine the approximate maximum seismic pore water pressure, The calculation was performed with the method explained in

Ref. 2, Chapter XII, Section 3.5, it was justified by means of a confrontation made with field seismic pore water pressure measurements in fine sand reported by Ishihara et.al, in Owi Island, Japan, Ref. 4 and 5.

The theoretical dominant period T_s of the sand deposit at the site, the maximum seismic relative horizontal soil displacements δ_{si}, and the apparent angle of internal friction during the seismic action are reported in Table I

TABLE I

SEC	DEPTH	HEIGHT	d	$\bar{\gamma}$	σ_{oi}	σ_{oc}	U_{sis}	δ_{sis}	μ_{sis}	ϕ_{sis}
1	0.5	10.75	1.0	0.85	0.430	0.275	0.187	1.589	163.61	10°.3
2	1.5	9.75	1.0	"	1.28	0.819	0.544	1.451	335.40	10°.8
3	2.5	8.75	1.0	"	2.13	1.363	0.869	1.305	485.10	11°.7
4	3.5	7.75	1.0	"	2.98	1.907	1.157	1.144	631.06	12°.7
5	4.5	6.75	1.0	"	3.83	2.451	1.406	0.978	777.72	13°.8
6	5.5	5.75	1.0	"	4.68	2.995	1.615	0.814	926.63	14°.9
7	6.75	4.50	1.25	"	5.74	3.674	1.821	0.576	1115.69	16°.4
8	8.25	3.00	1.50	"	7.01	4.486	1.996	0.357	1343.96	18°.1
9	9.75	1.50	1.50	"	8.29	5.306	2.097	0.158	1576.86	19°.8
10	11.25	0	1.50	"	9.56	6.118	2.130	0	1808.24	21°.4
	m	m	m	t/m^3	t/m^2	t/m^2	t/m^2	cm	t/m^2	

$\phi_d = 34°$, $\mu_{sis} = 756.45(\sigma_c - U_{sis})^{0.63}$ $T_s = 0.56$ sec

1 ton = 1000 Kg

PILE-SOIL INTERACTION

The second problem was to analyse the pile-soil interaction to determine the ratio of the free period of vibration of the pier T_p to the dominant period of vibration T_s of the sand deposit. With the value of T_p/T_s we determine the probable acceleration amplification at the deck of the pier, (Ref. 6 and 7). The value of T_p may be determined knowing the static horizontal deflection at the deck, therefore

$$T_p = 2\pi\sqrt{\frac{\delta_{st}}{g}} \qquad (4)$$

To estimate the value of T_p it was necessary to calculate the unit soil flexibility matrix and the unit pile flexibility matrix. These may be obtained assuming unit pile-soil horizontal reactions in so many horizontal sections as necessary for accuracy, Fig 3.

From conditions $X_i = +1$, Fig 4, we obtain $[\bar{S}_{ji}]$ the pile flexibility matrix and $[\bar{\delta}_{ji}]$ the soil flexibility matrix. Hence, the total horizontal displacements are

$$\{[\bar{S}_{ji}] + [\bar{\delta}_{ji}]\} \cdot |X_{ji}| \qquad (5)$$

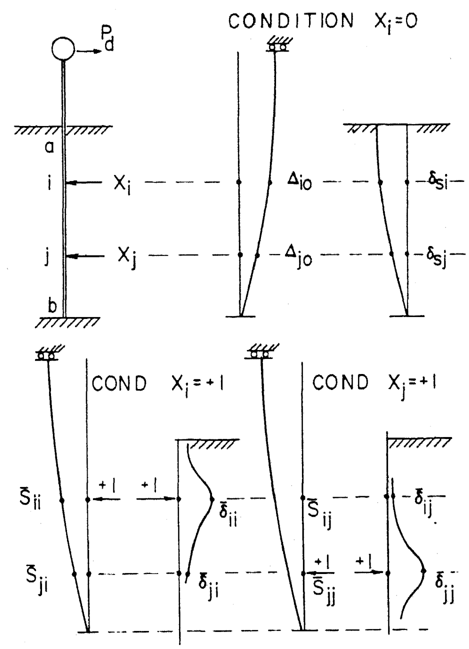


FIG 4. PILE-SOIL INTERACTION CONDITIONS

In which $|X_i|$ is the vector of the unknown horizontal reactions. Assuming $X_i = 0$, and the load per pile at the deck of 1.688 tons applied horizontally, we calculate the static deflections of the pile Δ_{io}, Fig 4. Therefore, we establish the matrix interaction equation for the static condition

$$\left[\bar{S}_{ji} + \bar{\delta}_{ji} \right] \cdot |X_i| = \Delta_{io} \qquad (6)$$

Solving this equation we determine the values of X_i, and thereafter the pile configuration by

$$\left[\bar{S}_{ij} \right] \cdot |X_i| - |\Delta_{io}| = |S_i| \qquad (7)$$

The maximum static deflection at the deck level is found to be 0.128 m. Therefore, the free period of vibration of the pier is approximately $T_p = 0.72$ sec, and $T_p/T_s = 1.29$. Using this value and assuming a fraction of critical damping of 5% we obtain an amplification factor on the order of three (Ref. 7). Therefore, the estimated dynamic maximum force at the deck elevation is $P_d = (3)\cdot(2)\cdot 1.69/9.81 = 1.034$ ton.

The dynamic behavior of the pier in its maximum amplitud is now calculated with $P_d = 1.034$ ton at the pier deck elevation and with the maximum horizontal soil displacements δ_{si} due to the seismic action given in Table I.

The following matrix equation may be established to investigate the seismic maximum response of the pier (*)

$$[\bar{S}_{ji} + \bar{\delta}_{ji}] \cdot |X_i| = |\Delta_{io} + \delta_{si}| \qquad (8)$$

The method in the application of equation (8) is iterative, because during seismic deformation the soil assumes a plastic condition at the upper sections. The pile and soil was divided in 10 sections as shown in Fig 3. The analysis indicated that the upper three sections enter into plastic condition with the following values:

Section 1	0.20 ton
Section 2	0.70 "
Section 3	1.10 "

The results of the final cycle of the pile-soil interaction calculations are shown in Fig 5, where the bending moments and deflection of the steel pipe are reported.The method used is called "HEMISES" it may be found in Ref. 2, Chapter XII, pp 567-588 or Ref. 7, Chapter IV.

From the results of the analysis reported it may be observed that the stresses of the steel pipe pile at the support of the deck increased on the order of 3450 kg/c^2 and at a depth of 8 mts into the sand deposit 2250 kg/c^2.

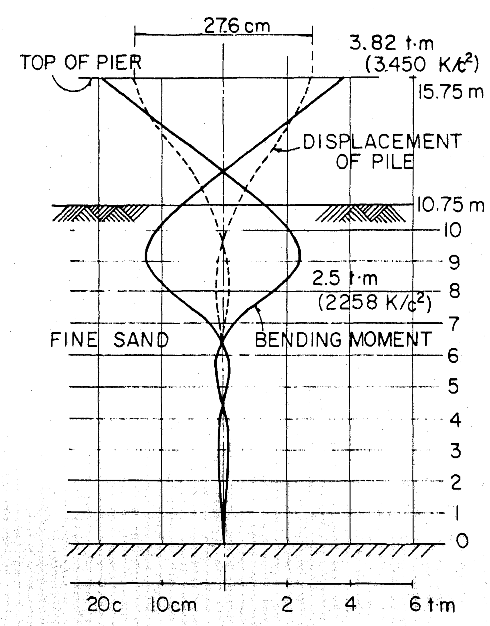


Fig 5. MAXIMUM AMPLITUDE SEISMIC BEHAVIOR OF STEEL PIPE PILES OF THE PIER

(*) The inertia of the mass of the pile is not considered

The elastic limit of the steel pipe material is on the order of 1900 kg/c^2 (27,000 lbs/in^2). Therefore, as the pipe pile reached these high stresses it was forced to yield, not recovering its original position. On the other hand, at this moment the free period of vibration of the pier T_p increased, and consequently the acceleration amplification at the deck elevation decreased, (Ref 2). The calculated double displacement amplitud of the pile head reached as a minimum 28 cm. Fig 5, showing a reasonable good agreement with the relative displacements and permanent distortions observed in the pier, Fig. 2.

CONCLUSIONS

A tentative interpretation of the seismic damage observed in this particular pier has been given based on the following working assumptions.

1) The maximum seismic pore water pressure in the soil is attained at the maximum ground surface acceleration of 200 gal.

2) The sand rigidity μ expressed by equation (2) based on the fine sand at the mouth of the Grijalva River is assumed to be valid at the site close to the mouth of the Coatzacoalcos River, for the same index properties.

3) The plastic forces in sections 1, 2 and 3 were estimated by the conventional plastic theory under instantaneous loading conditions. Deep sections show an elastic response.

It is recognized, however, that an "exact solution" cannot be obtained due to the approximate values of the parameters. Nevertheless, the analysis may be considered within the accuracy of engineering practice.

One may conclude, that following the method of analysis herein explained a designer could find that the size of the piles used were not suitable for the expected seismic ground surface acceleration. This method of analysis, however, was not known at the time these piers were designed and constructed.

In the present, the behavior of similar piers may be forcasted, and with a nominal factor of safety a safe design achieved.

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