

Tests of Steel Tubular Piles Driven Near Saigon River, Vietnam

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●POLITICALLY speaking, the Republic of Vietnam is very young, but geologically the country is very old. The central and northern portions consist of well-worn hills, small valleys and plateaus. The southern portion has been formed by the deposits of the Mekong, Saigon and Dong Nai Rivers.

One of the foreign aid projects of the U.S. Government consists of building 30 kilometers of new highway from the city of Saigon north. This highway crosses many smaller streams and canals as well as the two large rivers, the Saigon and the Dong Nai. Most of the highway alignment is through rice paddies between the many streams.

Core drillings showed quite a variation in subsoil characteristics. Foundation conditions were found to be good at the northern end of the highway with a good grade of rock at depths of less than 100 ft. At the southern end, no rock was found within the limit of the drilling equipment, which was 150 ft. The subsoil on the southern end consisted of varying depths of organic muck overlaying strata of clay and sand.

This subsoil condition dictated the use of piles for all structures. On the northern end, point-bearing piles of steel rolled sections were used. However, for the southern end it was apparent that if piles of high capacity with reasonable length were to be achieved, it would be necessary to use frictional displacement piles. This called for steel tubular piles of some type. Piles of steel WF sections were test driven to depths over 200 ft with only nominal driving resistance. The pier design selected for the smaller crossings was a multiple-pile bent. This substructure was to support 80-ft prestressed beams. The column heights required a minimum column diameter of 20 in. It was then decided to select a 20-in. tubular pipe pile for the substructure of these prestressed concrete beam bridges. A continuous plate girder was the design selected for the major 880-ft crossing of the Saigon River. This bridge was to incorporate 80-ft prestressed beam approach spans on pile trestle bents. For the major span, it was decided to use the 20-in. tubular pile or a fluted steel pile for supporting a reinforced concrete pier.

As there was no previous experience of piles of this size and type in this area, it was necessary to drive and test load several piles before the substructure design could be completed. For the Rach Chue crossing, a 4-span prestressed beam bridge, one test pile was deemed sufficient as borings indicated a uniformity of soil characteristics throughout the length of the structure. For the Saigon River, it was considered necessary to drive test piles near each bank with the idea of using a 20-in. tubular pile on the approach spans and a fluted pile under the main piers. One pile of each type was driven on both the north and south banks.

The fluted pile selected was a pile with the diameter varying from 8 in. at the tip to 18 in. in two 75-ft sections. Beyond the 75-ft length the pile diameter was constant at 18 in. The thickness of the pile was No. 3 gauge (0.239 in.).

DRIVING OF PILES

The driving of the Rach Chue pile was from the land near the edge of the stream and the Saigon piles were driven from a floating barge. All piles were driven with a McKiernan-Terry C-5 double acting steam hammer. This hammer has the following characteristics:

Weight of ram-piston in lb	5,000
Rated striking energy (ft lb per blow)	16,000

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Rated speed (strokes per min.)	110
Steam pressure, inlet, psi	100
Stoke-ins	18

The length of leads limited the maximum length of handling to one section. The 20-in. pile came in 40-ft sections and the 18-in. fluted pile in 38- and 36-ft sections. It was necessary during the driving of the piles to stop and weld the sections together. The 20- x 1/4-in. tubular pile required the welding of a collar at the top of the pile.

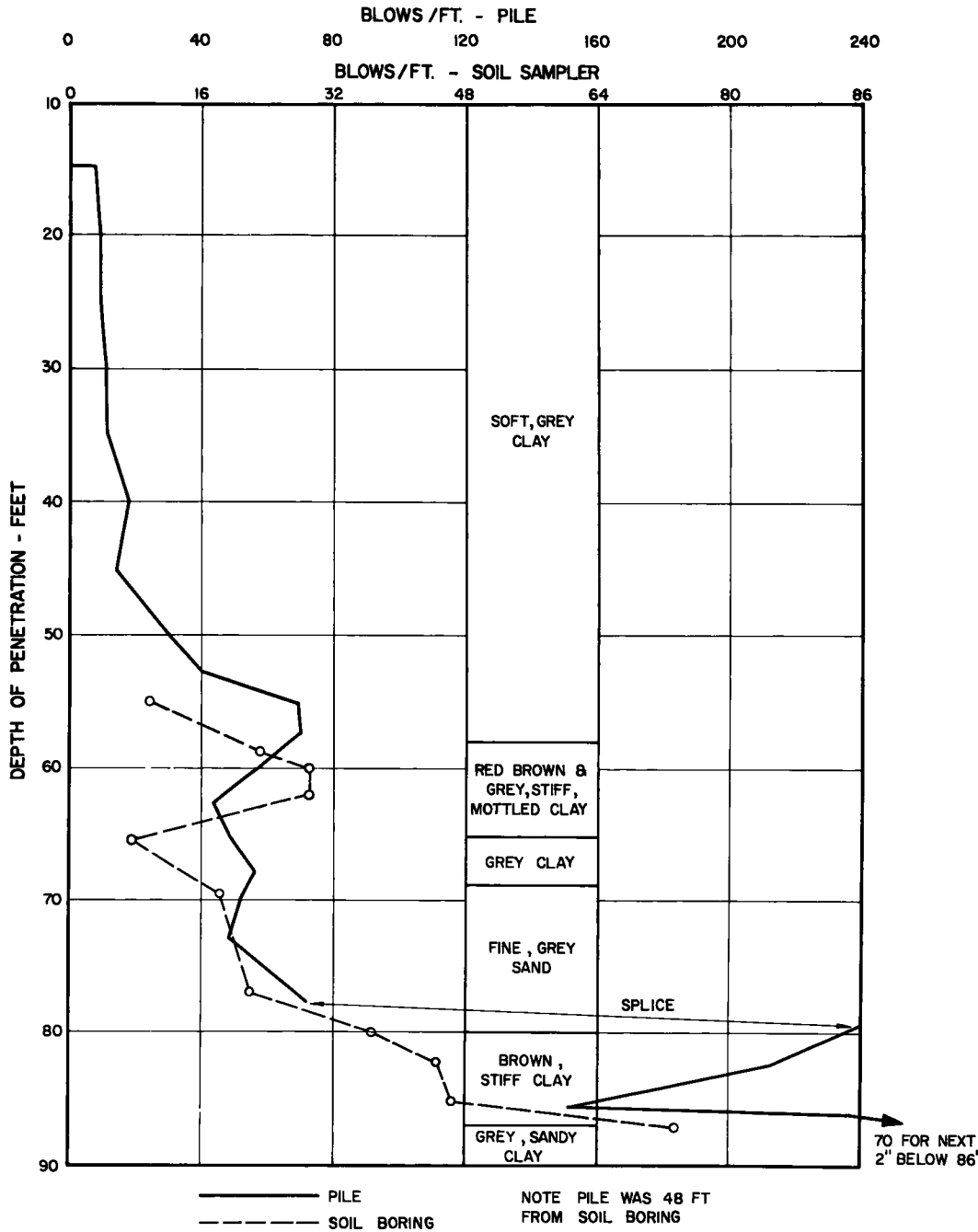


Figure 1. Location: Rach Chue River-20-in. x 1/4-in. steel pipe pile.

Without this collar, the end of the pile would curl during hard driving. This collar was cut off before the next section was welded on. The fluted pile drove as is, without any trouble. The piles were marked at 2-in. intervals for the counting of the blows per unit of penetration.

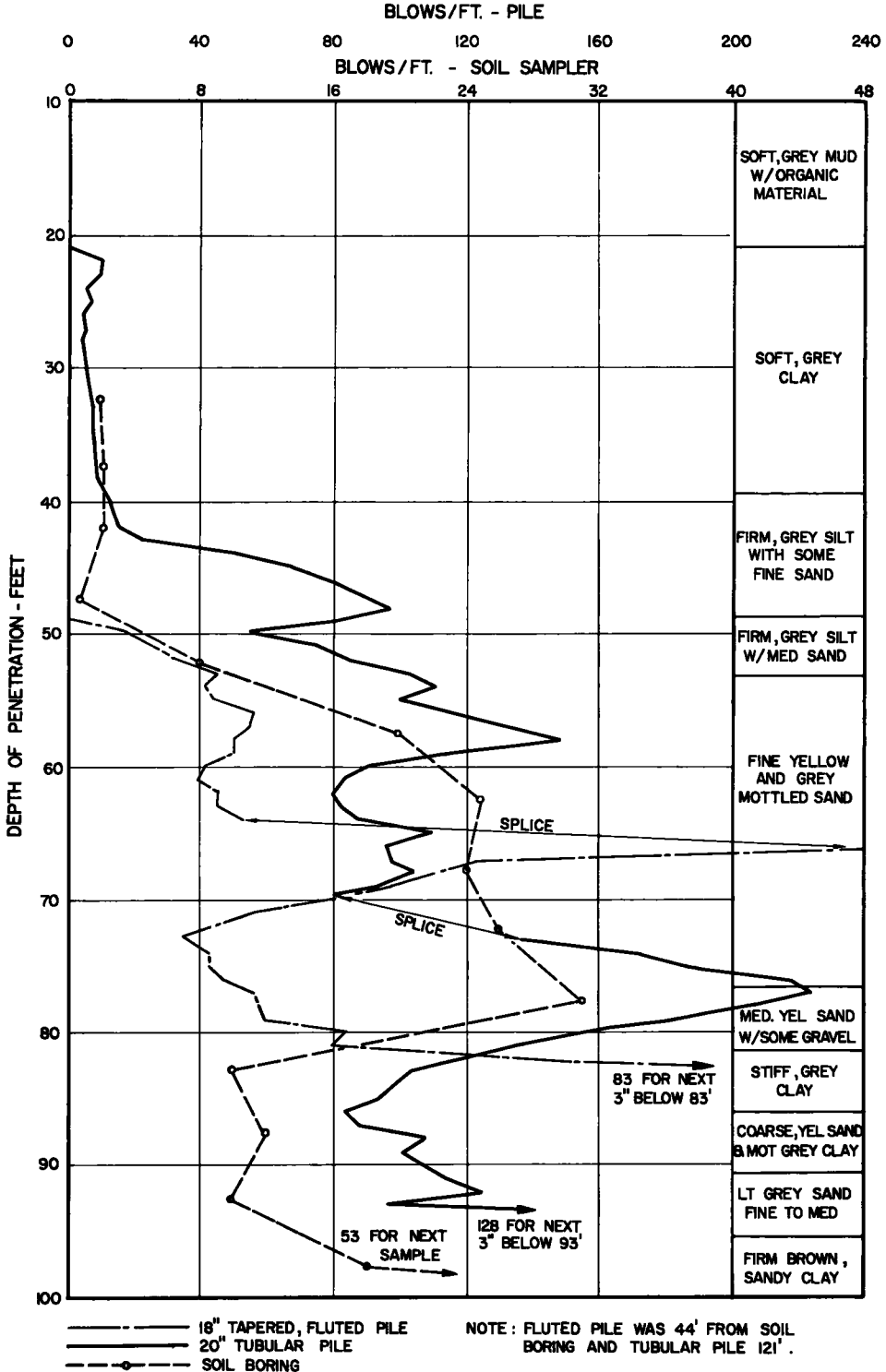


Figure 2. Location: Saigon River—north bank.

All the piles with the exception of the fluted pile on the south bank of the Saigon tightened up quite suddenly. Figures 1, 2 and 3 show the penetration-blow count relationship. Also shown are the blow counts that were necessary to drive the soil sampler.

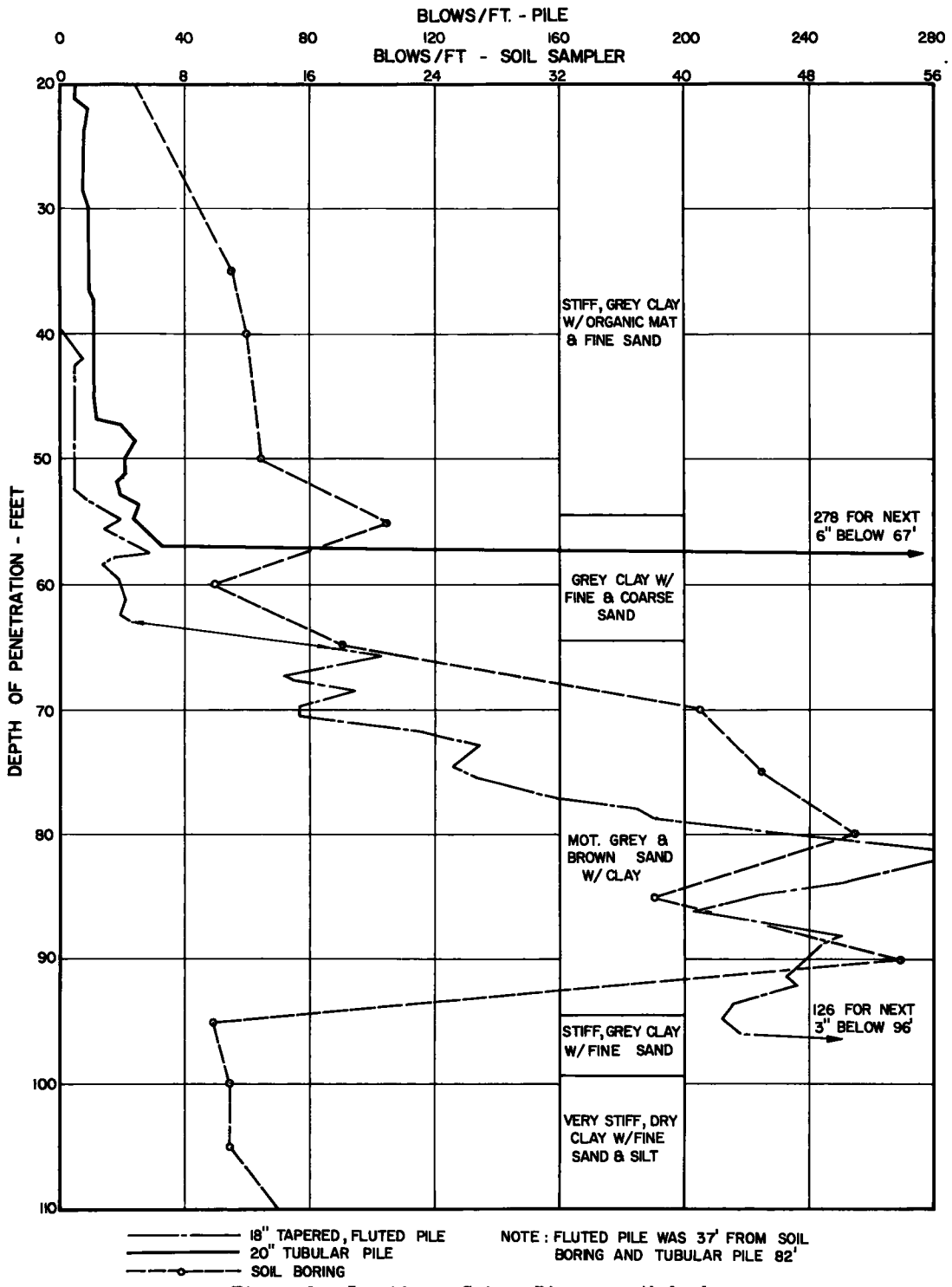


Figure 3. Location: Saigon River-- south bank.

After driving the piles, they were cut off at the required elevation and left to set several days and then filled with concrete.

LOAD TESTS

The pile was prepared for load tests by driving four timber piles forming the corners of a 16-ft square with the test pile in the center of the square. The timber piles were then connected with diagonal braces, and a horizontal grid of timber bracing near the top of the test pile gave lateral restraint to the test pile. On the north bank where the piles were farther out of the ground, cable tie-backs to concrete anchors were made to the timber piles.

A $\frac{1}{2}$ -in. steel collar was welded to the pile about 7 ft from the top, and a $\frac{3}{8}$ -in. loose collar was then slid down over the top of the pile to rest on the welded collar.

A loading platform consisting of a structural steel frame with a filling of reinforced concrete was then placed on the top of the test pile which had been ground smooth and level.

Diagonal braces consisting of a pair of 6- x 6- x $\frac{1}{2}$ -in. angles were then welded from the corners of the platform to the loose collar. The platform being 16 ft square could accommodate nine 5- x 5-ft concrete blocks in each layer. The weight of the platform was 20.2 tons. Figure 4 shows the platform in place on top of the pile.

The loads consisted of concrete blocks. Three sizes of blocks were used to give the necessary flexibility of loading. These sizes were 5 x 5 x 5 ft; $2\frac{1}{2}$ x 5 x 5 ft; and 5 x 5 x 3 ft.

While loads were being placed, screw-type jacks were placed on the top of the timber piles and jacked against the corners of the loading platform. In this manner the test piles were not required to carry any eccentric loads. All loads were balanced with respect to the pile before the corner jacks were retracted. Figures 4 through 7 show various stages of loading of the various piles.

The increments of load were generally as follows:

Increment	No. and Size of Blocks	Weight of Increment (tons)	Total Weight on Pile (tons)	Minimum Time Interval Between Increments (hr)
1	Platform	20.2	20.2	
2	3- 5 x 5 x 5	28.2	48.4	At least 24
3	2- 5 x 5 x 5	18.8	67.2	1
4	2- 5 x 5 x 5	18.8	86.0	2
5	2- 5 x 5 x 5	18.8	104.8	3
6	3- 5 x 5 x 5	28.2	133.3	12
7	2- 5 x 5 x 5	18.8	151.8	24
8	2- 5 x 5 x 5	18.8	170.6	24
9	2- 5 x 5 x 5	18.8	189.4	24-48
10	2- $2\frac{1}{2}$ x 5 x 5	9.4	198.8	24-48

Above the total load of 198.8 tons the increments were usually in 9.4 to 11.2 tons



Figure 4. Platform ready for loading—north bank of Saigon River.

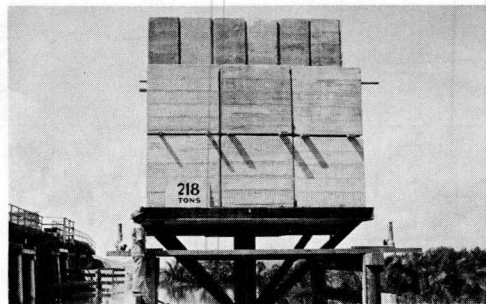


Figure 5. Final load on 20-in. tubular pile—Rach Chue.

(2 at $5 \times 5 \times 3$). Loads for increment 8 and above were usually placed in the morning and a settlement reading taken at that time. Another reading was taken in the evening and again the following morning. If there was noted a settlement between the afternoon reading and the following morning reading, no additional load was placed until settlement stopped. If no settlement had occurred between the stated time interval, an additional increment was then placed on the pile at the 24-hr interval.

Readings of settlement were taken with a Zeiss level, reading on a scale graduated in millimeters. The instrument was sufficiently close to the scale to be able to read to the nearest 0.5 mm (approximately $\frac{1}{50}$ in.). Reference bench marks were placed nearby and read at each reading of the pile. Figure 8 shows readings being taken of settlements. On the Rach Chue it was possible to set the instrument on firm ground. For the Saigon River piles it was not possible to do this and instrument platforms were constructed by driving timber piles near the test piles. These instrument platforms were to be utilized also during the construction of the actual bridge.

When the plot of the load vs settlement, relationship indicated that the limiting pile load was imminent, the pile was unloaded to measure the rebound. This unloading was usually conducted over an 8-hr time period. The pile was then left unloaded (except for platform weight) for a period of 16 hr. The loads were then reapplied in the same increments with approximately one hour between increments. The loadings beyond the previous high before unloading were then applied to the pile in accordance with the general

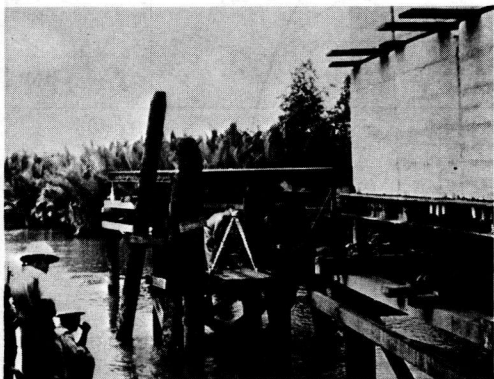


Figure 8. Reading settlements.

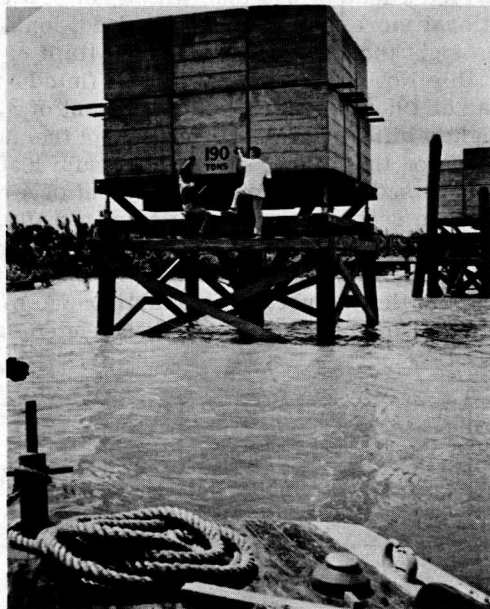


Figure 6. Loads on piles—north bank of Saigon River.

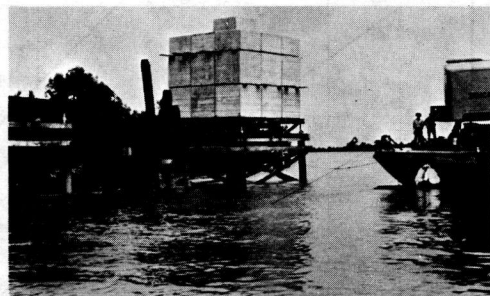


Figure 7. Load of 234 tons on 20-in. tubular pile—south bank of Saigon River.

procedure as above outlined.

In order to reduce the time of testing, two platforms were used on the Saigon River tests, and two tests were conducted at the same time.

RESULTS OF LOAD TESTS

The load vs settlement readings for each of the five piles is plotted in Figures 9 through 13. The general shape of these curves is similar, with a fairly straight-line relationship up to about 80 percent of the ultimate load and then a gradual curve beyond this point.

Pile No. 1 (20 in. at the Rach Chue)

carried a load of 208 tons with a net settlement of 4.0 mm (0.158 in.). With an additional increment of 10 tons an additional settlement of 4 mm occurred. Two hundred and eight tons was considered the limit of load.

Pile No. 2 (18 in. tapered and fluted—north bank of the Saigon River) carried a load of 199 tons with a net settlement of 3.0 mm (0.118 in.). This pile showed a slightly different reload curve from the unload curve. Two additional increments were placed on this pile to give a maximum load of 217.6 tons. This additional load of 18.6 tons caused an additional settlement of 4 mm. The loading was stopped at this point although settlement of the pile ceased after about 24 hr.

Pile No. 3 (20 in. at north bank of Saigon River) had a total settlement of 15.5 mm (0.61 in.) and a net settlement of 5 mm (0.197 in.) at a load of 254.3 tons. The load on this pile was not carried farther due to the limit in capacity of the load platform.

Pile No. 4 (18 in. tapered and fluted—south bank of Saigon River) had a net settle-

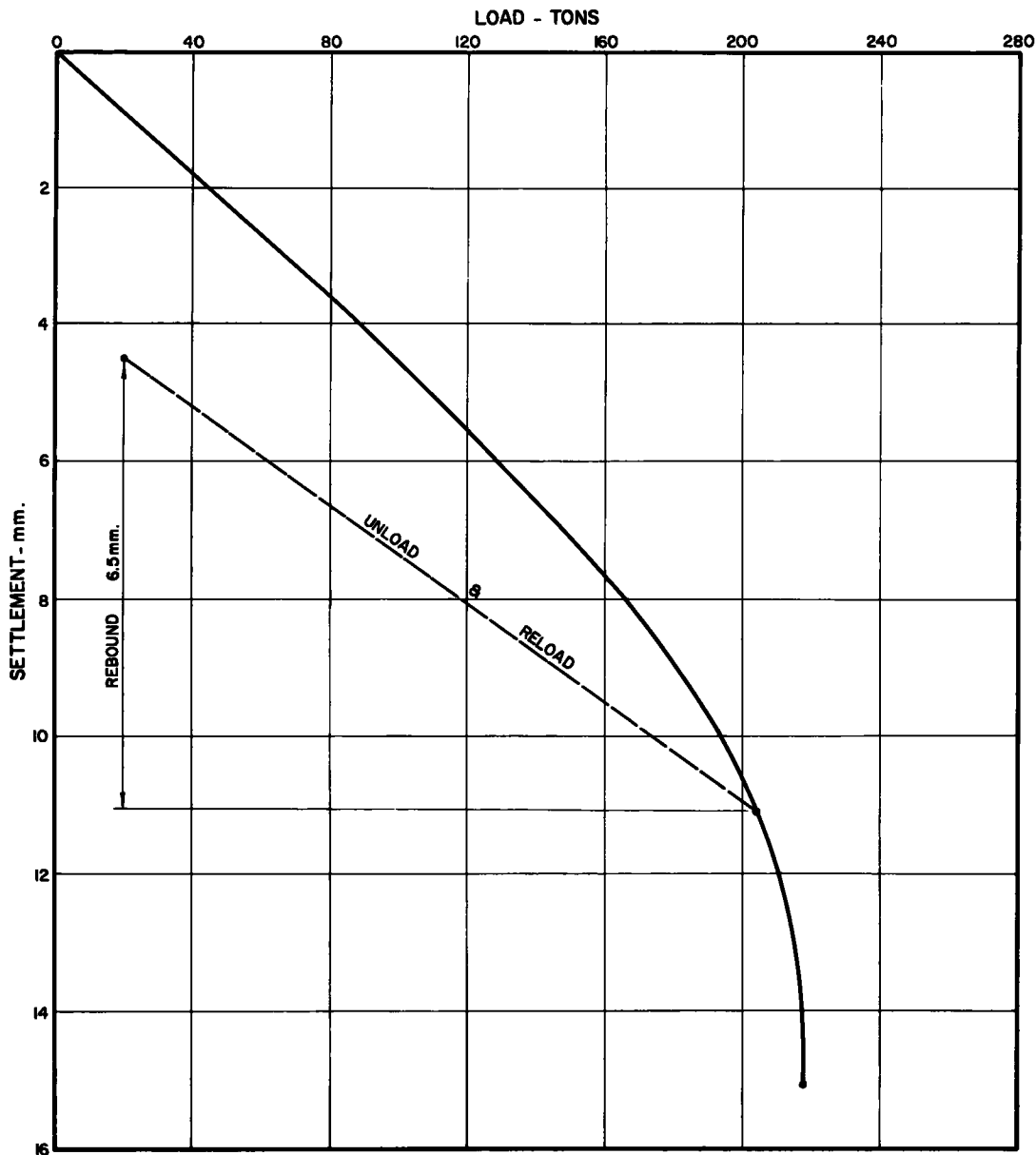


Figure 9. Rach Chue River—20-in. tubular pile.

ment of 7 mm (0.275 in.) at a load of 189.4 tons. Applying the AASHO limit of $\frac{1}{4}$ in net settlement, the limit load for this pile would be 180 tons.

Pile No. 5 (20 in. at south bank of Saigon River) had a net settlement of only 2.5 mm after a load of 189.4 tons. Using the same slope as the load-settlement curve, it can be said that the load could go over 200 tons before a permanent settlement of $\frac{1}{4}$ in. would result. The load was carried to 245.2 tons, but excessive settlement occurred in a 40-hr period at the last load increment of 11.2 tons. Large settlements at peak loads in all piles were very slow, therefore, no sudden failure occurred in any pile.

DISCUSSION

Sampler-Pile Blow Count Relationship

It can generally be said that the correlation between blows to drive the sampler and

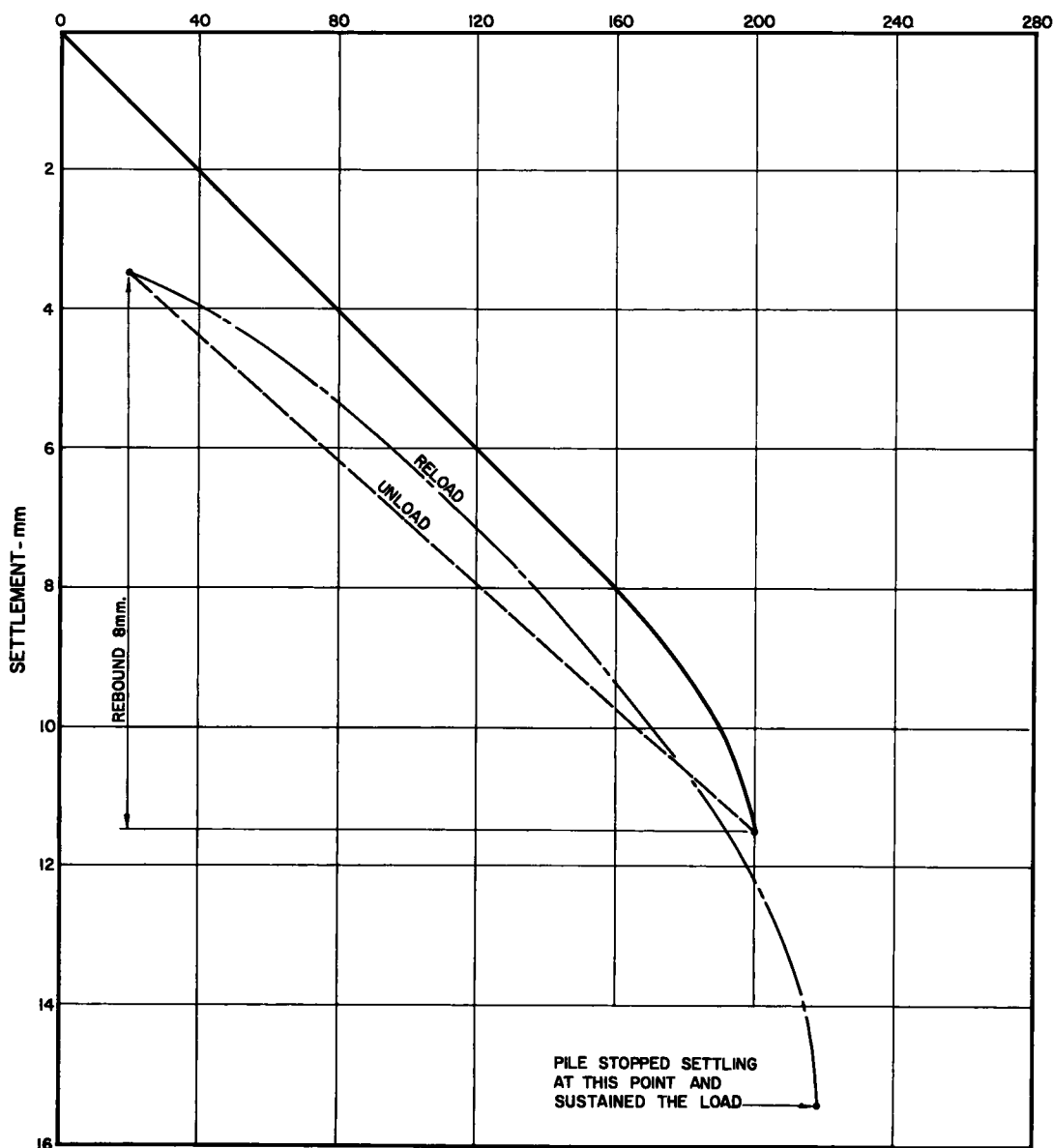


Figure 10. Saigon River--north bank 18-in. tapered, fluted pile.

blows to drive the pile was good. Very good correlation occurred for the Rach Chue pile. This was also true to a lesser extent for both the other two 20-in. tubular piles. The correlation was not as good for the fluted-taper piles. It is noted that the location of high blow count for both piles at the north bank of the Saigon River occurred at somewhat higher elevations than where the high blow count for the sampler occurred. This was also true for the 20-in. pile on the south bank. This could be due to the soil strata being actually a bit higher than as shown. The piles were some distance from the hole locations. Holes were bored at the final pier locations and test piles had to be moved a sufficient distance in order not to interfere with pier construction. Removal of the test piles is not contemplated before completion of the structure. In this type of delta deposit, it is not likely that the soil strata will have any appreciable slope. Results of the blow counts of the sampler give a basis for predicting penetrations of large displacement type of piles.

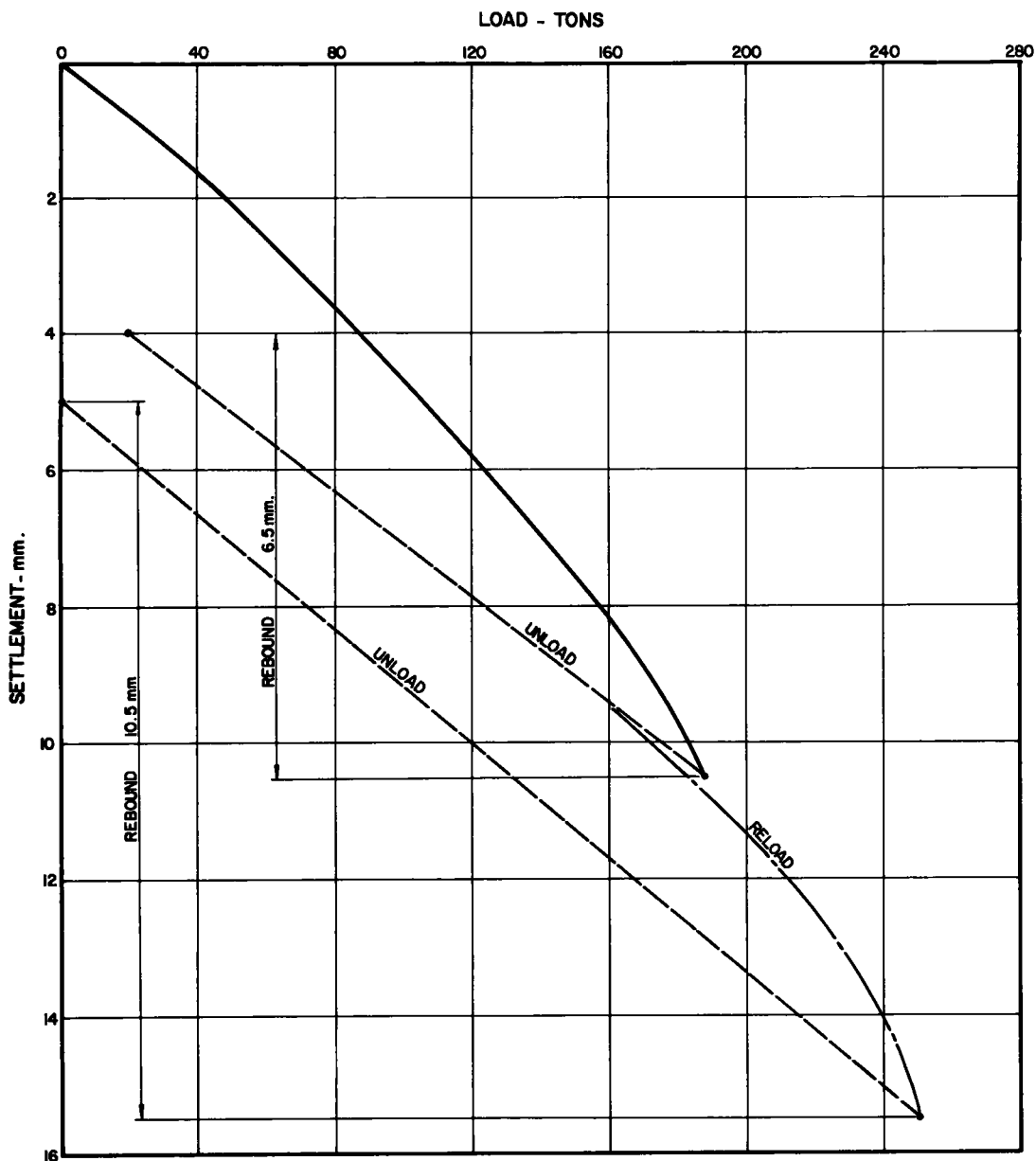


Figure 11. Saigon River— north bank 20-in. tubular pile.

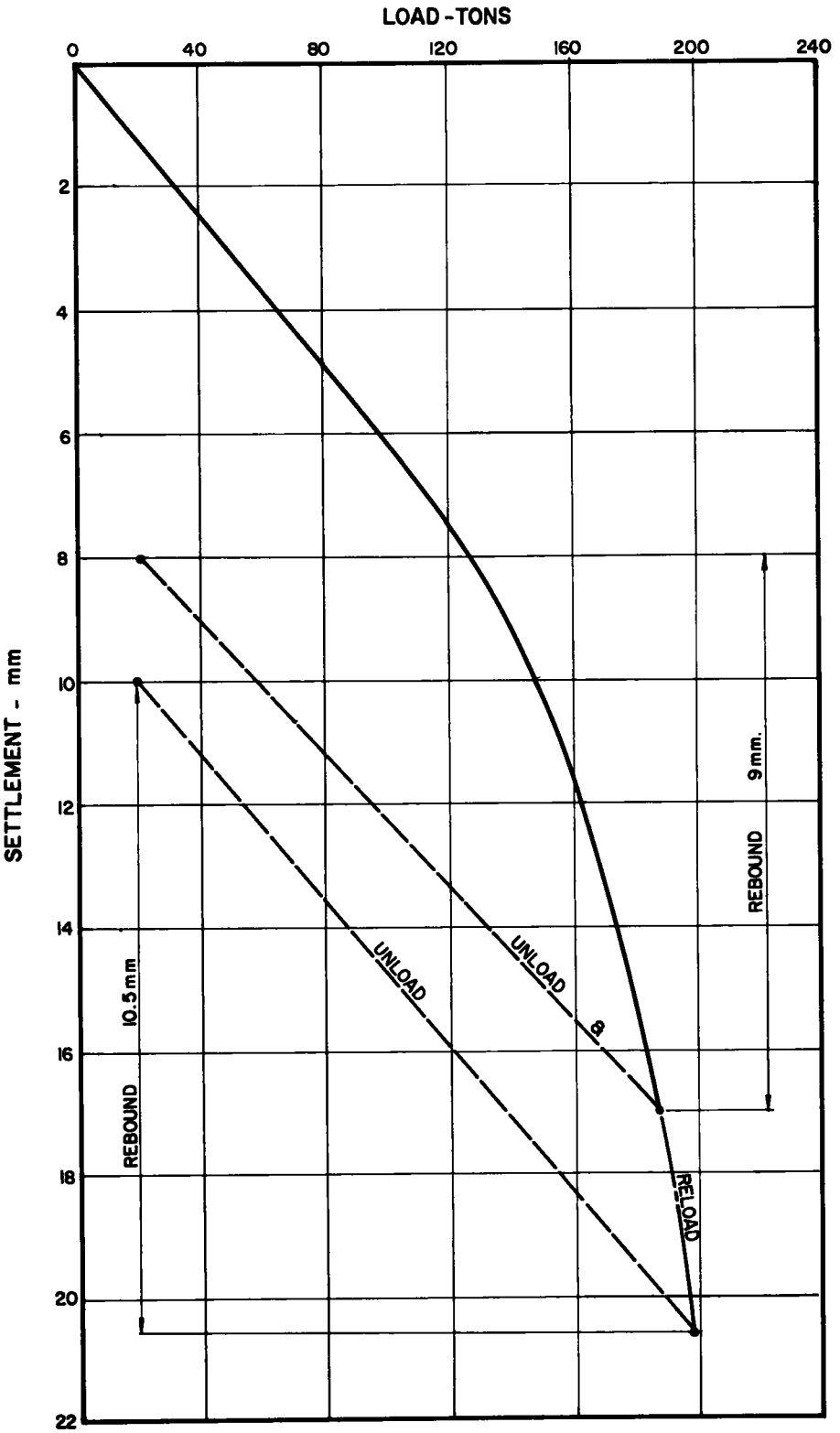


Figure 12. Saigon River— south bank 18-in. tapered, fluted pile.

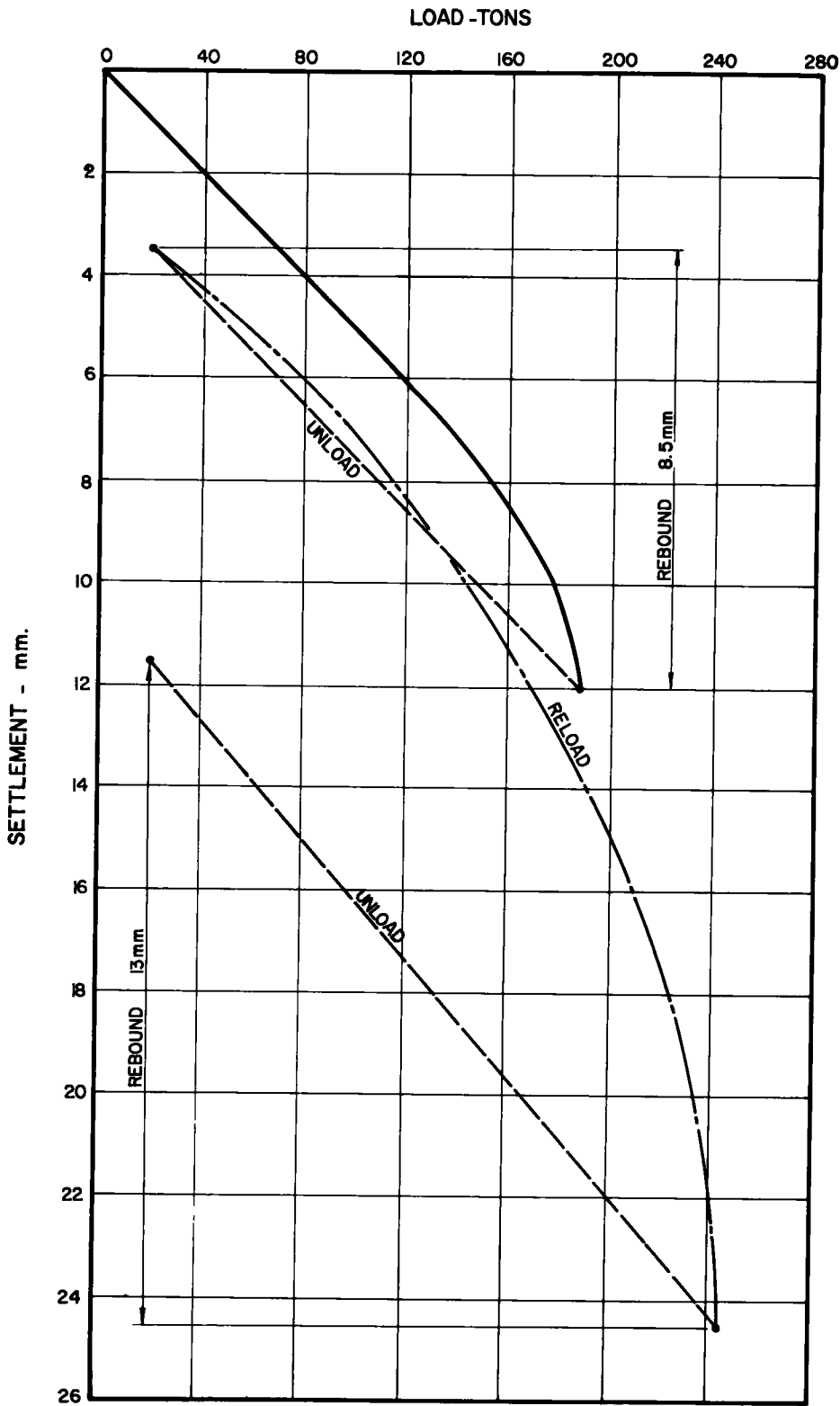


Figure 13. Saigon River-- south bank 20-in. tubular pile.

Penetrations

On the north bank, the two types of piles penetrated to nearly the same depth with the 20-in. tubular pile showing a slightly greater blow count. At the south bank, the fluted pile drove 29 ft deeper to obtain nearly the same blow count. The large 20-in. tip could not penetrate any appreciable distance into the layer of dense fine sand that started at about the 60-ft depth. The smaller 8-in. tip of the fluted pile provided no such strong resistance. This is verified by the blow count graph. The 20-in. pile showed a very sudden build-up in blow count while the 18-in. fluted pile showed a gradual build-up.

Comparison With Pile Formulas

A comparison of the loads carried by these five piles, with the dynamic pile formula of

$$R_u = \frac{12 E_f E_n}{S + \frac{1}{2} (C_1 + C_2 + C_3)} \times \frac{W_r + e^2 W_p}{W_r + W_p}$$

showed a considerable increase in load carried over the predicted capacity as obtained from the formula.

R_u = Ultimate carrying capacity of pile in pounds

E_n = rated energy of the hammer per blow in ft - lb

E_f = efficiency of the hammer

S = set of pile in inches per blow

C_1 = temporary compression allowance for pile head and cap in inches

C_2 = temporary compression of pile in inches

C_3 = temporary compression allowance for ground in inches

W_r = weight of ram - lb

W_p = weight of pile - lb

e = coefficient of restitution

The values of R_u from the dynamic formula were as follows:

Pile No. 1—Rach Chue, R_u = 280 kips

Pile No. 2—Fluted—north bank Saigon = 270 kips

Pile No. 3—20-in. tubular—north bank Saigon = 270 kips

Pile No. 4—Fluted—south bank Saigon = 270 kips

Pile No. 5—20-in. tubular—south bank Saigon = 336 kips

For all these piles, the constants C_2 and C_3 were so much greater in value than the set (S) that even an increase in S of double the value obtained would give practically the same value from the formula. The lesser weight of Pile No. 5 was responsible for the larger value of R_u as obtained by the formula.

The Modified Engineering News-Record formula gave values as follows:

Pile No. 1 = 252 kips

Pile No. 2 = 254 kips

Pile No. 3 = 215 kips

Pile No. 4 = 286 kips

Pile No. 5 = 302 kips

The actual factor of safety by using this formula would vary from a low of 1.26 for Pile No. 4 to a high of 2.3+ for Pile No. 3.

Because of the amount of sand through which the piles penetrated, a prediction of skin friction based on the shearing strength of the soil resulted in considerably smaller predicted loads than was actually obtained.

CONCLUSIONS

The following general conclusions can be drawn from these tests:

1. Large load capacity can be obtained from large diameter steel pipe piles driven deeply into sedimentary deposits of clay and sand.
2. The predicted capacity from the dynamic formula was quite low as compared to the actual load test.
3. The Modified Engineering News-Record formula will give too small a factor of safety for small penetrations of piles of this size.
4. Load tests should always be conducted when using piles of this size driven into clay and sand deposits.

Discussion

RALPH B. PECK, Professor of Foundation Engineering, University of Illinois—This paper presents the results of pile tests admirably suited for the design of the specific structures with which the author was concerned. Such tests may also provide valuable data for the general fund of knowledge concerning pile foundations. They represent a substantial financial investment and, with very little extra expenditure, could serve a broader purpose

In connection with the correlations among pile tests, the principal shortcoming of most records lies in the description of the properties of the soils surrounding the piles. The load-test procedures as such are usually satisfactory: the ASTM tentative standard method of test for "Load-Settlement Relationship for Individual Piles Under Vertical Axial Load" (D1143-57T) is quite suitable in all respects except that the procedure calls for no more than "a description of soil conditions at the location of the test pile."

The author presents the results of verbal descriptions and standard penetration tests. This is certainly a step in the right direction. It is suggested however, that more detail would be helpful. Unfortunately, the appropriate procedures differ for different soil conditions and hard-and-fast rules cannot be laid down. Nevertheless, the following would seem to be the minimum requirements:

For clays: verbal descriptions, natural water contents, liquid and plastic limits on representative samples, unconfined compressive strengths or equivalent measure of shear strength (as vane tests). Standard penetration tests provide useful supplementary data but are not in themselves adequate.

For silts: verbal, descriptions, standard penetration tests. Drained shear tests, if possible. Present knowledge regarding the properties of silts is very inadequate and no really satisfactory recommendations can be made.

For sands: verbal descriptions including estimates of size and grading, standard penetration tests on several borings near the test pile.

The records of pile load tests are often either too brief, or more elaborate than necessary. A form found suitable in studying the numerous tests included in the survey of friction piles for the Highway Research Board is that shown in HRB Special Report 36. A single page was found adequate for most records.

D. ALLAN FIRMAGE, Closure — Peck has pointed out a major difficulty in correlations among pile tests. That is, the desired quantity of soil information is in most cases somewhat lacking. This is true because most load tests have the primary objective of determining pile capacity for use in a specific design and the gathering of information on a research basis is only incidental to the work. This was the case with regard to the pile load tests in Vietnam. The main objective was to determine pile capacities for the foundation of bridges in the specific locations.

Soil borings had been taken at the proposed pier locations and laboratory tests conducted on the samples. As the test piles could not be incorporated in the final structure or easily removed, the test piles were driven some distance away from the pier locations. Because of the nature of the geology in this region, it was considered that prediction of pile capacity in the actual structure could quite accurately be obtained from the test piles. Figures 2 and 3 of the paper showed that the test piles on the south bank of the Saigon River were 37 ft and 82 ft from the soil boring and on the north bank

TABLE 1
LOCATION: SAIGON RIVER—SOUTH BANK

Depth-Meters From	To	Class- ification	Un. Comp. Str. Kg/M ²	Dry Density Kg/L	Moisture Content (%)
0	3.60		No sample—muck		
3.60	5.12	Clay	1,200	0.794	86.1
5.12	6.64	Clay	2,650	0.833	80.9
6.64	8.17	Clay	3,610	0.884	73.1
8.17	9.70	Clay	2,650	0.836	81.0
9.70	11.22	Clay	2,970	0.872	68.8
11.22	12.74	Sandy clay	4,250	0.853	75.1
12.74	14.26	Clay	5,540	0.877	68.6
14.26	15.79	Clay	3,290	0.865	74.6
15.79	17.31	Sandy clay	2,490	1.174	45.5
17.31	18.84	Sandy clay	3,610	1.088	54.6
18.84	20.36	Sand	1,363	1.808	15.6
20.36	21.88	Sand	3,370	1.792	18.3
21.88	23.41	Sand	2,810	1.735	20.4
23.41	24.93	Sand	3,450	1.744	19.4
24.93	26.46	Sand	2,490	1.770	22.9
26.46	27.98	Sand	3,610	1.750	18.8
27.98	29.50	Clay	8,587	1.131	52.0
29.50	31.03	Clay	8,427	1.141	51.8
31.03	32.55	Clay	10,433	1.157	50.0

TABLE 2
LOCATION: SAIGON RIVER—NORTH BANK

Depth-Meters From	To	Class- ification	Un. Comp. Str. Kg/M ²	Dry Density Kg/L	Moisture Content (%)
0	6.89	Soft muck—no sample			
6.89	8.42	Clay	3,300	0.789	90.3
8.42	9.94	Clay			
9.94	11.47	Clay	1,440	0.822	81.9
11.47	12.99	Silty clay	5,300	0.837	80.7
12.99	14.51	Sand	2,650	0.903	74.6
14.51	16.04	Sand	13,240	1.478	29.5
16.04	17.56	Sand	6,020	1.880	14.4
17.56	19.09	Sand			
19.09	20.61	Sand		1.770	19.3
20.61	22.13	Sand			
22.13	24.65	Silty sand			
24.65	25.18	Sand	1,565	1.715	21.4
25.18	26.71	Clay	6,020	1.256	40.4
26.71	28.23	Sandy clay	26,900	1.616	26.0
28.23	29.75	Sandy clay	2,650	1.814	16.5
29.75	31.28	Sand	1,850	1.891	13.9
31.28	32.80	Clay	6,980	1.895	15.2
32.80	34.32	Clay	5,580	1.894	15.0

44 ft and 121 ft from the nearest boring. The results of laboratory analysis of samples from these two borings is given as part of this discussion (Table 1 and 2).

It would have been desirable to have made additional borings near the test piles but because of a limited number of drill rigs in proportion to the quantity of drilling necessary for the over-all program additional borings at these locations could not be justified.

Peck's outline of information requirements in pile load tests should be followed wherever practical. This outline should be an aid to those conducting future pile tests.