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LATERAL LOAD TESTS ON PILES IN BRIDGE EMBANKMENTS

Presented at the 53rd Annual Meeting
of the Highway Research Board
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STATE OF CALIFORNIA

BUSINESS AND TRANSPORTATION AGENCY

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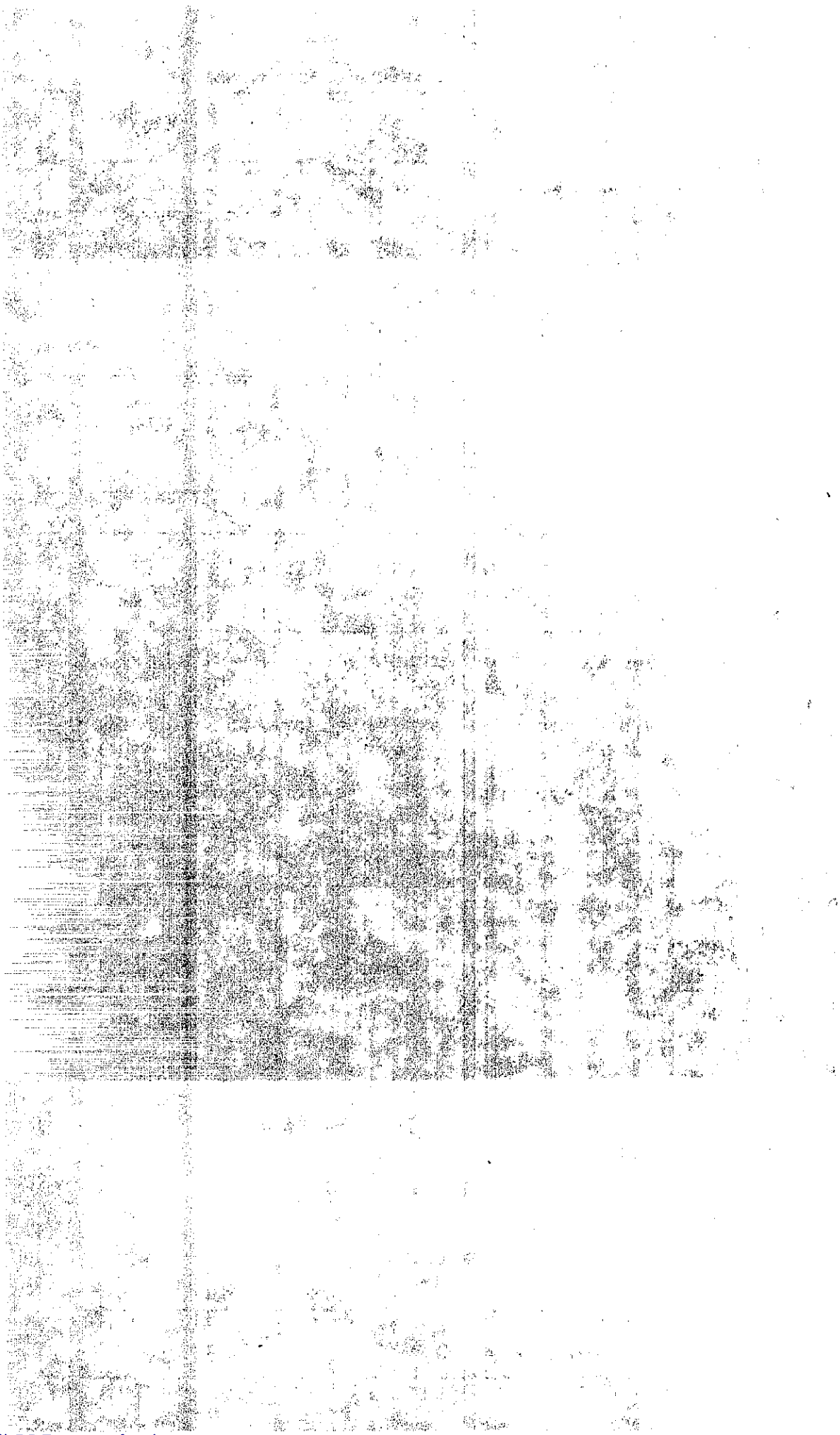

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LATERAL LOAD TESTS ON PILES IN BRIDGE EMBANKMENTS

By

Joseph A. Paduana
Wilfred S. Yee

Presented at the 53rd Annual Meeting
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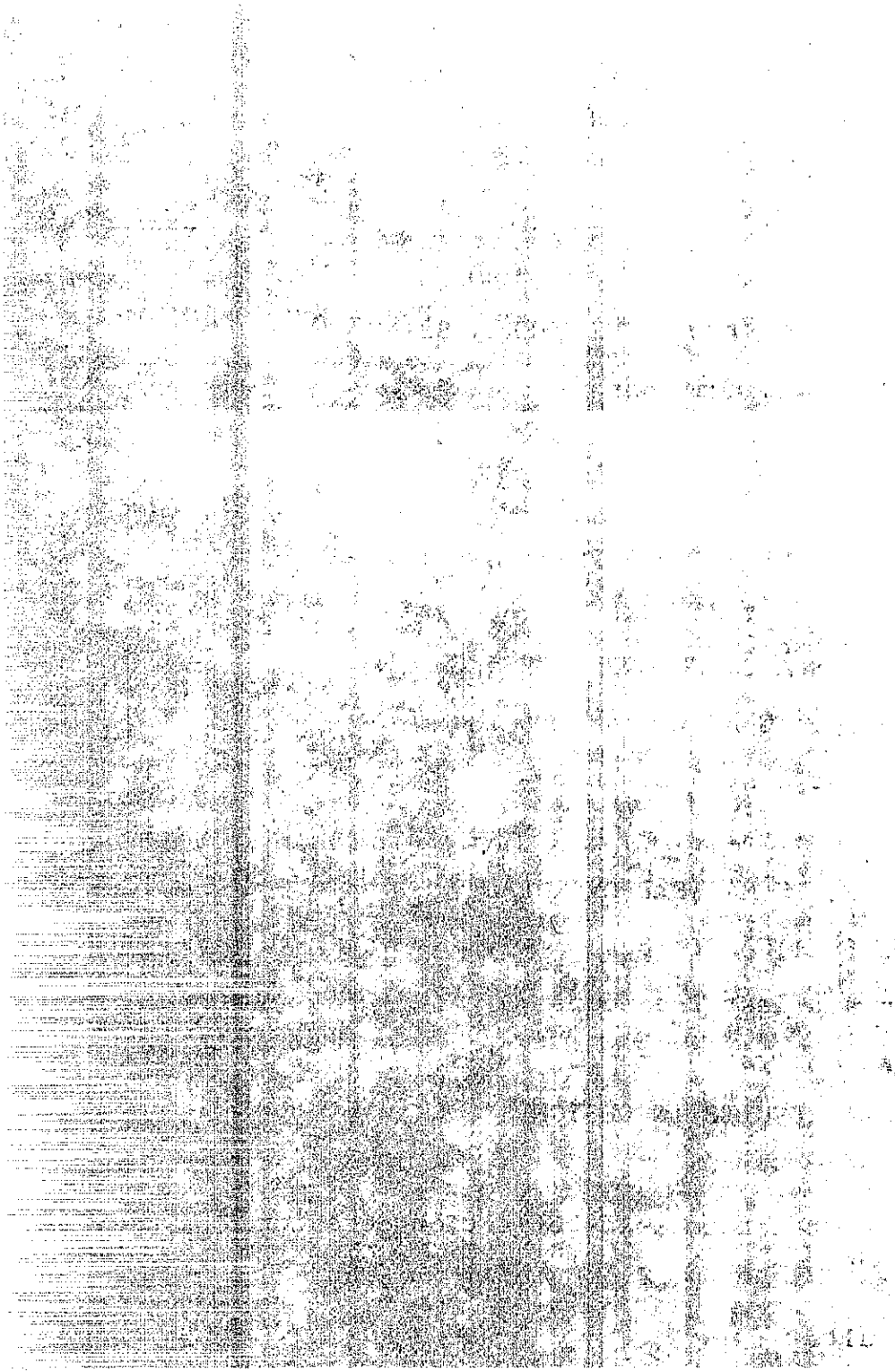

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LATERAL LOAD TESTS ON PILES IN BRIDGE EMBANKMENTS

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INTRODUCTION

Piles in bridge foundations are often required to resist lateral forces resulting from post-tensioning of concrete superstructures, seasonal changes in length of superstructures without expansion joints, earth pressures, and earthquakes. It is important, therefore, to be able to predict the behavior of piles in bridge embankments subjected to lateral forces to ensure that both the deflections and stresses of the soil-pile-structure system are within tolerable limits.

Most analytical techniques for evaluating the behavior of a pile subjected to lateral loads are based on the theory of a beam on an elastic foundation (1) and require solutions of the differential equation

$$EI \frac{d^4y}{dx^4} + ky = 0 \qquad (1)$$

where EI is the flexural stiffness of the pile, y is the lateral deflection of the pile at depth x below the ground surface (Fig. 1a), and k is the subgrade modulus, a measure of the stiffness of the soil surrounding the pile. By definition, k is expressed in terms of y as

$$k = -\frac{p}{y} \qquad (2)$$

where p is the soil reaction across the width of the pile, which develops as a result of the pile deflection. As defined,

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k has units of force per unit length per unit of deflection.

The solution of equation (1) depends on the assumption regarding the variation of k with depth. Solutions are available (2) for any fixed variation of k with depth, but generally required the use of a computer. For many practical problems, solutions are readily available for the cases where k is either constant with depth or increases linearly with depth in accordance with the expression

$$k = n_h x \tag{3}$$

where n_h is the constant of horizontal subgrade reaction. Terzaghi (3) suggested that for preloaded cohesive soils, k may be assumed constant with depth. Davisson and Gill (4), however, showed that this assumption may lead to underestimates of moment and deflections by a factor of 2. Terzaghi further proposed that for sands, k should vary linearly with depth from a value of zero at the surface, and the argument for such a variation was further developed by Reese and Matlock (5). Table 1 shows recommended values of k as summarized by Davisson (6), based on the literature and experience.

Solutions of equation (1) are readily available in nondimensional form. For the case where $k = n_h x$, Reese and Matlock (5) prepared a complete set of charts for determining deflections, moments, shears, and soil pressures. For the case of constant k or stepped variation in k, Davisson and Gill (4) prepared similar solutions for deflections and moments. The selection of appropriate magnitudes and distribution of k-values is basic to these solutions.

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While the preceding investigations have led to a good understanding of the behavior of laterally loaded piles, further experimental information is desirable for the conditions shown in Figure 1b of a typically laterally-loaded pile in a bridge embankment. As shown, the embankment pile is inserted into an 18" (46 cm)-diameter predrilled hole in the compacted fill and driven into the underlying natural soil. The annular space between the pile and fill is backfilled with pea gravel to reduce negative skin friction resulting from future vertical settlements. Experimental evidence on the behavior of laterally loaded piles or on values of n_h or k for these soil conditions are very limited.

Full-scale lateral load tests on embankment piles were performed at three sites in northern California (Fig. 2). The purposes of the testing program were: (1) to determine the lateral load-deflection behavior of individual vertical embankment piles; (2) to correlate the results of full-scale tests on embankment piles with theoretical solutions; and (3) to determine experimental values of n_h (or k) for such embankment conditions. All test piles were used in the construction of bridge structures.

LATERAL LOAD TESTS AT OCCIDENTAL DRIVE OVERCROSSING

Test Layout and Soil Conditions

The location of test piles at the Occidental Drive Overcrossing in Sacramento, California, is shown in Figure 3a. Lateral load tests were performed on 6 of the 12 vertical piles for bridge Abutment 3. All piles were considered as single isolated piles

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not influenced by adjacent piles. The properties of the 10 3/4" (27.3 cm) O.D. steel pipe test piles are given in Table 2. The piles are embedded in approximately 28 ft (8.5 m) of the underlying natural sandy silt and in approximately 12 ft (3.7 m) of the annular pea gravel and surrounding compacted silty sand with gravel (Fig. 3c).

Figure 3b shows a general boring log for the site, including values of the standard penetration resistance. The fill was compacted to a minimum of 95 percent of maximum density based on the California Compaction Test 231-E. No groundwater was encountered during the subsoil investigation. Properties of samples taken in the compacted fill and in the underlying natural soil are given in Table 3. Figure 4 shows the grain-size distribution of the pea-gravel backfill. More detailed descriptions of the soil profile are given in Ref. (7).

Instrumentation

Lateral loads were applied normal to the pile axis with a calibrated hydraulic jack at the ground surface. The corresponding lateral deflections at the top of the piles were measured with deflection transducers.

All test piles were equipped with SR-4 electrical resistance strain gages along the embedded upper portion of the piles. Strain observations were recorded under lateral load to measure flexural stresses within the pile. The flexural stresses were then converted to bending moments. Complete details of the instrumentation are given in Ref. (7).

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Test Procedure

Three types of lateral load tests were performed at Occidental (Fig. 3a): Test A on Piles 2 and 3, Test B on Piles 10 and 11, and Test C on Piles 2, 3, 6, 7, 10 and 11. Tests A and B were conducted before construction of the abutment, while Test C was conducted after construction of the superstructure. Therefore, test piles were considered as free-head for Tests A and B and fixed-head for Test C.

The lateral load for Tests A and B was applied at the ground line by jacking horizontally between a pair of test piles. During Test A, a constant incremental lateral deflection was induced between the tops of Piles 2 and 3, allowing the applied lateral load to vary, in order to maintain a constant deflection. Constant total incremental deflections of 1/2, 1, 1-1/2, and 2 inches (1.3, 2.5, 3.8, and 5.1 cm) were induced between the piles and maintained for 2 hours for each increment, in sequence. During Test B, constant incremental lateral loads of 5, 10, 15, 20 and 25 kips (22, 44, 67, 89, and 111 kN) were applied, in sequence, between Piles 10 and 11. Each load increment was held constant for a minimum period of 2 hours and until the pile head movement was less than 0.010 inches (0.025 cm) per hour. The test procedure for the after-construction Test C is presented later.

Free-head Test Results and Analysis

The free-head test results (Tests A and B) are plotted in Figure 5a as lateral load applied at the ground line versus

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deflection at the top of the pile for Piles 2, 3, 10, and 11. The corresponding strain gage readings were converted to moments at the selected load levels, and these measured moments versus depth are plotted as solid lines in Figure 6 for the 4 test piles.

The results of the free-head tests were analyzed by means of the nondimensional solution (5), using theoretical expressions and nondimensional coefficients based on a linear variation of k with depth, equation (3). For this solution, a relative stiffness factor T is defined as

$$T = \sqrt[5]{EI/n_h} \tag{4}$$

For a free-head pile, the horizontal deflection produced by a lateral load P at the ground line is given as

$$y = A_y \frac{PT^3}{EI} \tag{5}$$

where A_y is the free-head depth coefficient. The corresponding bending moment is given as

$$M = A_M PT \tag{6}$$

where A_M is the free-head moment coefficient. Values of A_y and A_M are readily obtained from charts (5) for various depths along the pile.

Values of n_h were determined in the following manner. A trial value of n_h was assumed for an applied lateral load, and the corresponding deflection at the top of the pile was computed,

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Equation (5), and compared with the measured deflection. When the computed and measured deflections agreed closely, the assumed value of n_h was considered satisfactory. Plots of n_h - values versus deflection at the top of the piles are shown in Figure 5b. To verify the assumption regarding the distribution of subgrade modulus, theoretical moments were computed, equation (6), using values of n_h from Figure 5b. The theoretical moments are shown in Figure 6 as dashed lines for comparison with the measured moments from strain gage readings for the same lateral load. The agreement is reasonably good for practical purposes.

After-construction Test Results and Analysis

For the after-construction Test C (Fig. 3a), the effects of post-tensioning and dead load of the bridge superstructure were considered as loads for the test piles. The total prestress force for the concrete superstructure was 9,200 kips (41,000 kN).

The lateral deflections at the top of the test piles and the strains along the piles were measured after post-tensioning of the superstructure, but with the falsework still in place. The measured deflections are shown in Figure 7a. The strain gage readings were converted to moments, and plotted versus depth for Piles 6 and 7 as shown in Figures 7b and 7c. (It is noted that Piles 2, 3, 10, and 11 were subjected to a reloading in Test C in a direction normal to the initial loadings in Tests A and B. The test results and analysis (7) for the reloading are not included, as only one cycle loadings are considered herein.) Construction operations did not permit measuring deflections after the falsework was removed, although the strains along the

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test piles were recorded. These strains were also converted to moments and plotted versus depth for Piles 6 and 7 in Figure 7b and 7c.

The results of the after-construction tests were also analyzed using the same nondimensional solution (5), but assuming fixed-head conditions. The deflection of a pile fixed against rotation at the ground line is given as

$$y_F = F_y \frac{PT^3}{EI} \tag{7}$$

where F_y is the fixed-head deflection coefficient, and P is the shear force at the ground line. The corresponding bending moment is given as

$$M_F = F_M \, PT \tag{8}$$

where F_M is the fixed-head moment coefficient. Values of F_y and F_M are also readily obtained from Charts (5) for various depths along the pile.

Values of n_h were determined as follows. Due to post-tensioning of the bridge superstructure with the falsework in place, a lateral force and a moment were induced at the top of each pile in the abutment. The lateral force was unknown, but the moments could be determined from strain gage readings (Fig. 7). Assuming a trial value of n_h, a lateral force P corresponding to the measured moment at the pile top was computed from Equation (8). Substituting this value of P in Equation (7), a theoretical deflection at the top of the pile was computed and compared

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with the measured deflection (Fig. 7a). When the theoretical deflection agreed closely with the measured deflection, the assumed value of n_h was considered satisfactory. The value of n_h computed in this manner for Piles 5 and 6 is 28 lbs/in^3 (7.6 N/cm^3). The effect of the sloped embankment normal to the abutment was not considered and may account for the somewhat lower value of n_h compared to the free-head test values (Fig. 5b).

Because construction procedures prevent the measurements of lateral deflections after the falsework was removed, it was not possible to compute theoretical moments for comparison with measured moments for this condition. However, it was possible to compute the lateral force and deflection at the pile tops (for the after-falsework-removed condition) using the corresponding measured moments (Fig. 7), the value of n_h determined as 28 lb/in^3 (7.6 N/cm^3), and Equations (7) and (8). The values thus computed for Piles 6 and 7 are P equal to 11.2 kips (49.8 kN) and y_F equal to 0.170 inches (0.432 cm).

LATERAL LOAD TESTS AT LINDA CREEK OVERCROSSING

Test Layout and Soil Conditions

A plan of the test piles at Abutment 4 of the Linda Creek Overcrossing near Roseville, California, is shown in Figure 8a. Lateral load tests were performed on 4 of the vertical H-piles; the properties are given in Table 2. The embedment of a typical test pile in the underlying natural silty clay to sandy silt and

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in the annular pea gravel surrounded by an existing compacted silty clayey sand embankment is shown in Figure 8c. The embankment had been compacted to a minimum of 90% of maximum density based on the California Compaction Test 231-E. A log of the nearest boring is shown in Figure 8b. Table 3 lists the properties of samples taken in the compacted fill and in the underlying natural soil. A more detailed description of the soil profile is given in Ref. (8).

Instrumentation and Test Procedure

Lateral loads were applied by jacking horizontally between a pair of test piles before construction of the abutment. The instrumentation for measuring deflections, loads, and bending strains along the pile was similar to that described for the Occidental project. In addition, the slope at the top of the test piles was measured with an electronic level (8).

Only constant incremental lateral load tests (similar to the procedure described for Test B at Occidental) were performed: Test A on Piles 1 and 2 and Test B on Piles 3 and 4 (Fig. 8a). Lateral loads were applied in increments of 5 kips (22 kN). It should be noted that H-pile 4 rotated slightly during driving. As a result, the lateral load was not applied in the plane of the principal axis of Pile 4 (8).

Test Results and Analysis

A plot of lateral load applied at the ground line versus deflection at the top of the pile is shown in Figure 9a for Piles 1, 2, 3, and 4.

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The results of the tests were evaluated using the same procedure described for the free-head tests at Occidental. A plot of n_h-values versus deflection at the top of the piles is shown in Figure 9b. Theoretical moments along the test piles and theoretical slopes at the pile tops were also computed (8) using these n_h-values and the nondimensional solution (5). Maximum theoretical moments and maximum measured moments compared (8) within 15 percent for the same lateral load for Piles 1, 2, and 3. However, the maximum theoretical moment for H-pile 4, which had rotated during driving, was about 30% greater than the maximum measured moment. Theoretical slopes at the top of the test piles also agreed reasonably well with the measured slopes (8), except for Pile 4.

LATERAL LOAD TESTS AT NORTH MAXWELL OVERHEAD

Test Layout and Soil Conditions

The location of Test Piles 2, 3, 9, and 10 at the North Maxwell Overhead near Maxwell, California, is shown in Figure 10a. The properties of the 10 3/4" (27.3 cm) O.D. steel pipe piles are listed in Table 2. As shown in Figure 10c, a typical test pile is embedded in approximately 50 ft (15 m) of the underlying natural soft to stiff clay (with sand at deeper depths) and in approximately 20 ft (6 m) of the annular pea gravel and surrounding compacted silty sandy clay fill. The fill was compacted to a minimum of 90% of maximum density based on the California Compaction Test 231-E. A log of the nearest boring is shown in Figure 10b and includes values of the standard penetration resistance. Properties

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of samples taken in the compacted fill and in the underlying natural soil are given in Table 3.

Instrumentation and Test Procedure

Lateral loads were applied by jacking between test piles before construction of the abutment. The instrumentation was similar to that described for the Occidental test piles and in addition, included an electronic level for measuring the slope at the top of the test piles.

The test procedure for Test A on Piles 2 and 3 and Test B on Piles 9 and 10 are, respectively, similar to the constant-incremental-lateral deflection test and the constant-increment-lateral load test described for the Occidental project.

Test Results and Analysis

Test results for Piles 2, 3, 9 and 10 are plotted in Figure 11a as lateral load applied at the ground line versus deflection at the top of the pile. The results were analyzed using the same procedure described for the free-head tests at Occidental. Values of n_h determined by this procedure are plotted versus deflection in Figure 11b. Using these values of n_h and the nondimensional solution (5), theoretical moments along the test piles and theoretical slopes at the top of the piles were also computed (8). A comparison (8) between maximum theoretical moments and maximum measured moments showed agreement within 8 percent. Similar good agreement resulted from a comparison (8) of theoretical and measured slopes at the pile tops.

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SUMMARY AND USE OF TEST RESULTS IN DESIGN

n_h - Values

A summary of the test results for the three sites is given in Table 4 and includes n_h - values for single cycle of loading. Values of n_h for the three embankments varied overall from 10 to 65 lb/in^3 (2.7 to 17.6 N/cm^3), with higher values for the stiffer embankment. The results suggest the following simple approximate relationship between values of n_h in lb/in^3 and the standard penetration resistance for the pile-soil conditions studied herein:

$$n_h \approx \bar{N} \tag{9}$$

where $\bar{N}$ is the average blows/ft from standard penetration tests within the depth of the compacted embankment.

It seems reasonable to expect some rough correlation between values of n_h and N values. As the "pressure bulb" (seat of settlement) for lateral deflections of a pile includes both the pea-gravel backfill and the surrounding compacted fill, values of n_h should depend on the relative density of the pea-gravel and the stiffness of the compacted fill. But for each test pile, the pea-gravel backfill was placed in the same manner by dumping from an extended chute of a truck. Therefore, differences in n_h - values would be accounted for primarily in differences in the stiffnesses of the compacted fill, as indicated by the N values.

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Effect of End Restraint on Load-deflection Behavior

Figures 5a and 7 show respectively lateral load versus lateral deflection relationships for the free-head and fixed-head pile conditions at Occidental. Because of the end restraint and base friction contributed by the pile abutment, the fixed-head pile resists approximately twice as much lateral load as the free-head pile at the same lateral deflection of 0.25 inches (0.64 cm).

Effective Length

The effective length of a laterally loaded pile is defined as the length of the embedded portion of a pile which is effective in transferring lateral load from the pile to the soil. The lower portion of a pile beyond the effective length shows no appreciable deflection and thus, no appreciable soil reaction. For practical purposes, the effective length may also be considered as the depth to the first zero-deflection point of a pile.

The range of effective length for the test piles at the three sites was computed for lateral loads within a practical range and is presented in Table 5. The effective length varied from 8 to 12 ft (2.4 to 3.7 m), which lies within the depth of the compacted fill. Thus, the compacted fill of a bridge embankment provides the major support for a laterally loaded pile, and the influence of the underlying natural deposit is practically negligible.

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Use of Test Results

The tests reported herein provided data that was used in the design of pile foundations for bridge embankments. The test results are in terms of the behavior of a single pile under single cycle of loading. Cyclic loading and pile spacing in the direction of the load must also be considered in selecting an appropriate value of the subgrade modulus k. Cyclic loading has the effect of reducing the value of k (9, 10). Pile spacing less than eight pile diameters in the direction of the load also has the effect of reducing the value of k (11).

CONCLUSIONS

The following conclusions are considered valid from an analysis of full-scale lateral load tests on embankment piles at three bridge sites in northern California.

1. Comparisons of measured and theoretical bending moments versus depth for vertical piles in bridge embankments show that a linear variation of horizontal subgrade modulus with depth ($k = n_h x$) is a reasonable approximation.
2. Values of n_h for single cycle loading at three embankment sites varied overall from 10 to 65 lb/in^3 (2.7 to 17.6 N/cm^3), with higher values for the stiffer embankment. Test results suggest a simple rough approximation, i.e., $n_h \approx \overline{N}$, for the soil-pile conditions studied herein.
3. Limited data indicate that a fixed-head pile resists approximately twice as much lateral load as a free-head

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pile at the same lateral deflection of 0.25 inches (0.64 cm).

4. The computed effective length (depth to first zero-deflection point of pile) of the embankment piles varied from 8 to 12 ft (2.4 to 3.7 m) for lateral loads between 9 to 16 kips (40 to 71 kN).

5. A compacted fill of 12 ft (3.7 m) or more provides the major support for a laterally loaded pile, and the influence of the underlying natural deposit is practically negligible.

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6. Davisson, M. T. Lateral Load Capacity of Piles. Highway Research Record 333, 1970, pp. 104-112.

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9. Alizadeh, M., and Davisson, M. T. Lateral Load Tests on Piles - Arkansas River Project. Proc. ASCE, Vol. 96, No. SM5, Sept. 1970, pp. 1583-1604.

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TABLE I (6)

ESTIMATED VALUES FOR k

SOIL TYPE	VALUE
GRANULAR SOILS	n_h RANGES FROM 1.5 TO 200 LB/IN.^{3}, IS GENERALLY IN THE RANGE FROM 10 TO 100 LB/IN.^{3}, AND IS APPROXIMATELY PROPORTIONAL TO RELATIVE DENSITY
NORMALLY LOADED ORGANIC SILT	n_h RANGES FROM 0.4 TO 3.0 LB/IN.^{3}
PEAT	n_h IS APPROXIMATELY 0.2 LB/IN.^{3}
COHESIVE SOILS	k IS APPROXIMATELY 67 C_U, WHERE C_U IS THE UNDRAINED SHEAR STRENGTH OF THE SOIL.

NOTE: THE EFFECTS OF GROUP ACTION AND REPEATED LOADING ARE NOT INCLUDED IN THESE ESTIMATES.

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TABLE 2

PROPERTIES OF TEST PILES

TEST SITE	PIPE NO.	PILE TYPE	MOMENT OF INERTIA OF STEEL PILE OR H-PILE ($IN.^4$)	MOMENT OF INERTIA OF CONCRETE IN PIPE PILE ($IN.^4$)	MODULUS OF ELASTICITY OF CONCRETE $\times 10^3$ ($KIPS/IN^2$)	EI $\times 10^6$ OF PILE ($KIPS-IN.^2$)
OCCIDENTAL	2,3,6,7, 10 & 11	10-3/4" O.D. STEEL PIPE[a] (1/4" WALL) FILLED WITH REINF. CONC.[b]	114	542	4.30[c] & 4.66[c]	6.00 & 6.19
LINDA CREEK	1 & 2	H-PILE[a] (8" X 8", 40 LB/FT)	146	—	—	4.38
LINDA CREEK	3 & 4	H-PILE[a] (10" X 8", 42 LB/FT)	205	—	—	6.15
NORTH MAXWELL		10-3/4" O.D. STEEL PIPE[a] (1/4" WALL) FILLED WITH CONCRETE	114	542	4.33[c] & 4.45[c]	5.76 & 5.82

[a] MODULUS OF ELASTICITY OF STEEL TAKEN AS 30×10^3 KIP/IN.^{2}

[b] CONCRETE REINFORCED WITH 4-#6 VERTICAL BARS

[c] MODULUS OF ELASTICITY OF CONCRETE DETERMINED BY CALIFORNIA TEST METHOD 522: LOWER VALUE FOR PILE TEST A & B, UPPER VALUE FOR SUBSEQUENT PILE TEST C.

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TABLE 3

PROPERTIES OF SAMPLED SOIL FOR TEST PILES

LOCATION / SOIL PROPERTY	OCCIDENTAL		LINDA CREEK		NORTH MAXWELL	
	COMPACTED FILL	UNDERLYING NATURAL SOIL	COMPACTED FILL	UNDERLYING NATURAL SOIL	COMPACTED FILL	UNDERLYING NATURAL SOIL
DEPTH BELOW GROUND SURFACE, FT.	6 TO 10	18 TO 20	5 TO 7	15 TO 17	5.5 TO 10.5	25.5 TO 30.5
UNIT WEIGHT, LB/FT^3	122	114	128	121	125	124
GRAIN SIZE:						
% GRAVEL	8[a]	0[b]	5	2	0	1
% SAND	45[a]	39[b]	45	4	6	7
% SILT	35[a]	52[b]	29	41	31	33
% CLAY	12[a]	9[b]	21	43	63	59
AVG. WATER CONTENT, %	23	18	22	29	24	28
LIQUID LIMIT, %	22[a]	24[b]	30	44	50	48
PLASTIC LIMIT, %	2[a]	2[b]	16	17	20	19
SHEAR STRENGTH PARAMETERS (UU TRIAXIAL COMPRESSION)						
RANGE OF CONFINING PRESSURES, TONS/FT^2	NOT AVAILABLE		1/4 TO 1	1/2 TO 2	1/4 TO 1	1/2 TO 2
ϕ_U, DEGREES			8.0	9.5	13.5	7.0
C_U, LB/FT.^{2}			1400	400	1400	1900

[a] SAMPLE TAKEN AFTER CONSTRUCTION, JULY 11, 1973, AT A DEPTH OF 5 FT. BELOW GROUND SURFACE.

[b] SAMPLE TAKEN AFTER CONSTRUCTION, JULY 11, 1973, AT A DEPTH OF 17 FT. BELOW GROUND SURFACE.

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TABLE 4

SUMMARY OF TEST RESULTS

TEST LOCATION	TEST PILE	PILE TYPE	RESTRAINT AT TOP OF PILE	DEPTH OF PEA-GRAVEL BACKFILL (FT)	COMPACTED FILL		RANGE[c] OF n_h (LB/IN.^{3})
					GENERAL SOIL TYPE	DEPTH (FT)	$\bar{N}$[b] (BLOWS/FT)
OCCIDENTAL	2,3,10 & 11	10-3/4" O.D. STEEL PIPE FILLED WITH REINF. CONC.	FREE-HEAD	12	SILTY SAND WITH GRAVEL	12	30	30-65
OCCIDENTAL	6 & 7	DITTO	FIXED-HEAD[a]	12	DITTO	12	30	28
LINDA CREEK	1 & 2	H-PILE (8" X 8" 40 LB/FT)	FREE-HEAD	12	FINE SILTY CLAYEY SAND	16	19	15-20
LINDA CREEK	3 & 4	H-PILE (10" X 8" 42 LB/FT)	FREE-HEAD	12	DITTO	16	19	15-40
NORTH MAXWELL	2,3,9 & 10	10-3/4" O.D. STEEL PIPE FILLED WITH CONCRETE	FREE-HEAD	20	SILTY SANDY CLAY	20	22	10-30

[a] ASSUMED AS FIXED-HEAD AFTER CONSTRUCTION OF SUPERSTRUCTURE
[b] AVERAGE BLOWS/FT, STANDARD PENETRATION TESTS
[c] FOR SINGLE CYCLE OF LOADING

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TABLE 5

EFFECTIVE LENGTH (DEPTH TO FIRST ZERO-DEFLECTION POINT OF PILE)

TEST LOCATION	TEST PILE	RESTRAINT AT TOP OF PILE	DEPTH OF COMPACTED FILL (FT)	RANGE OF LATERAL LOAD AT TOP OF PILE (KIPS)	RANGE OF DEPTH TO FIRST ZERO-DEFLECTION POINT OF PILE (FT)
OCCIDENTAL	2, 3, 10 & 11	FREE-HEAD	12	13-15	8-9
OCCIDENTAL	6 & 7	FIXED-HEAD	12	11	12
LINDA CREEK	1, 2, 3, & 4	FREE-HEAD	16	15-16	9-10
NORTH MAXWELL	2, 3, 9 & 10	FREE-HEAD	20	9-10	10-12

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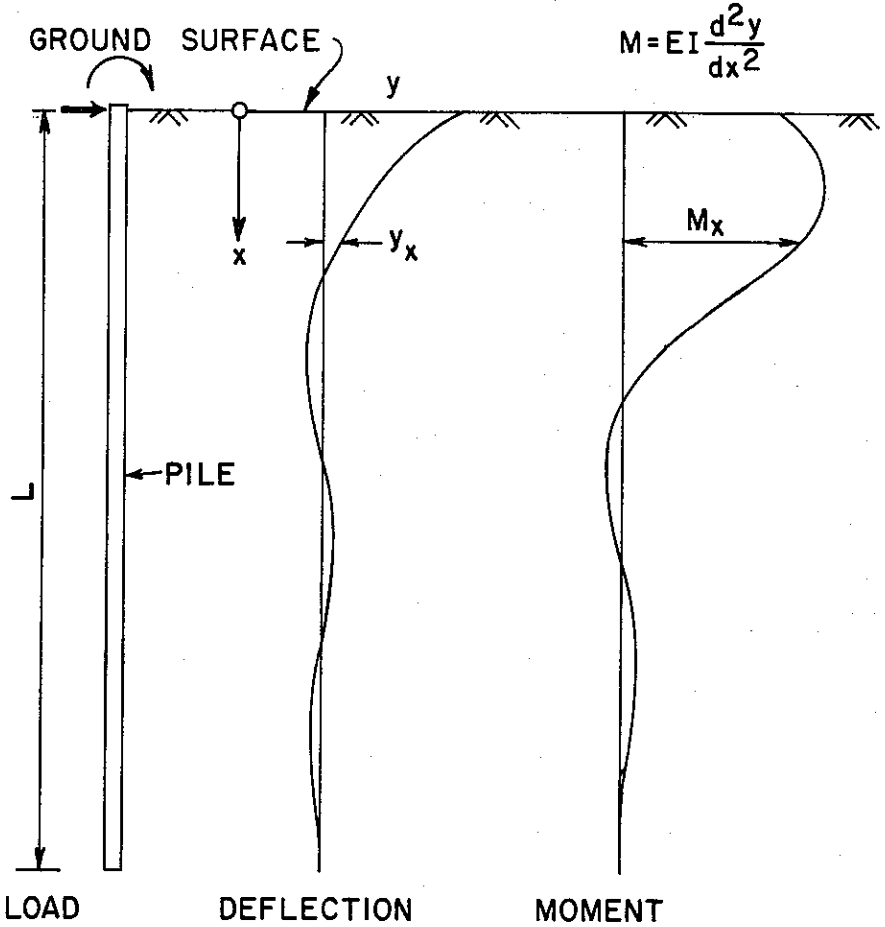
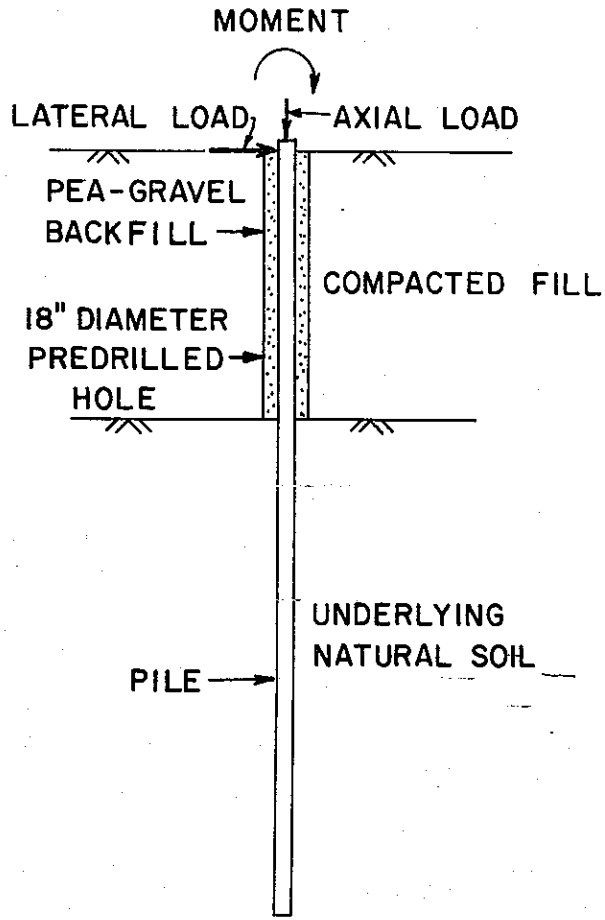


(a) TYPICAL DEFLECTION AND MOMENT



(b) EMBANKMENT PILE

FIGURE 1

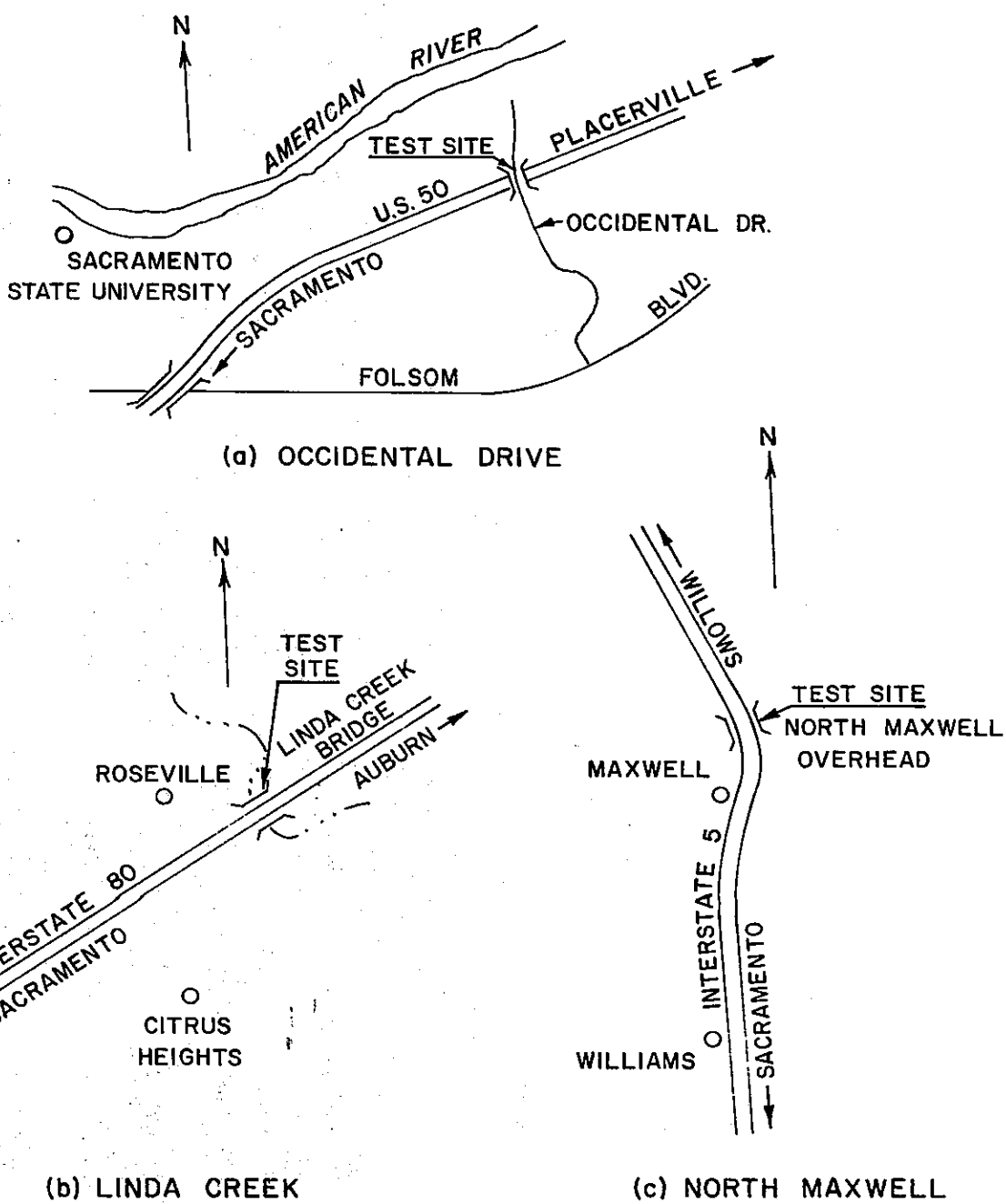


FIGURE 2

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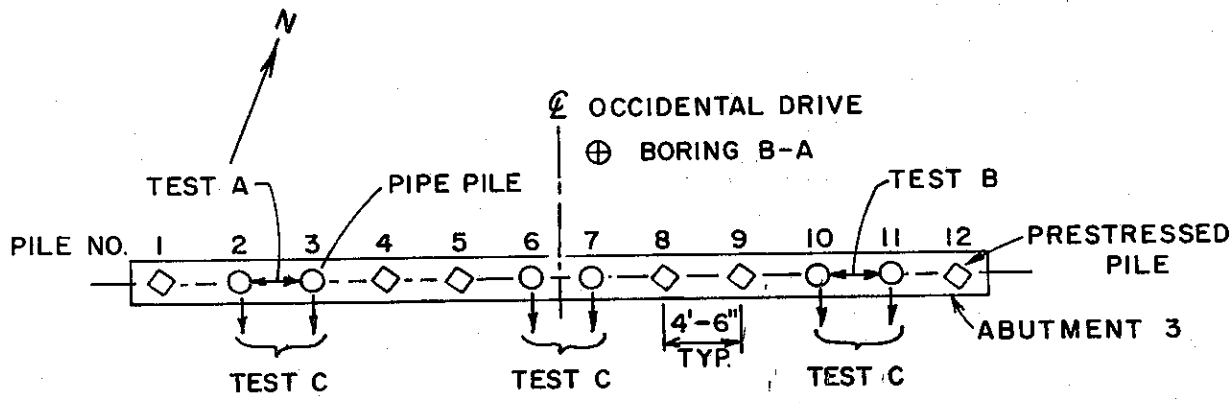
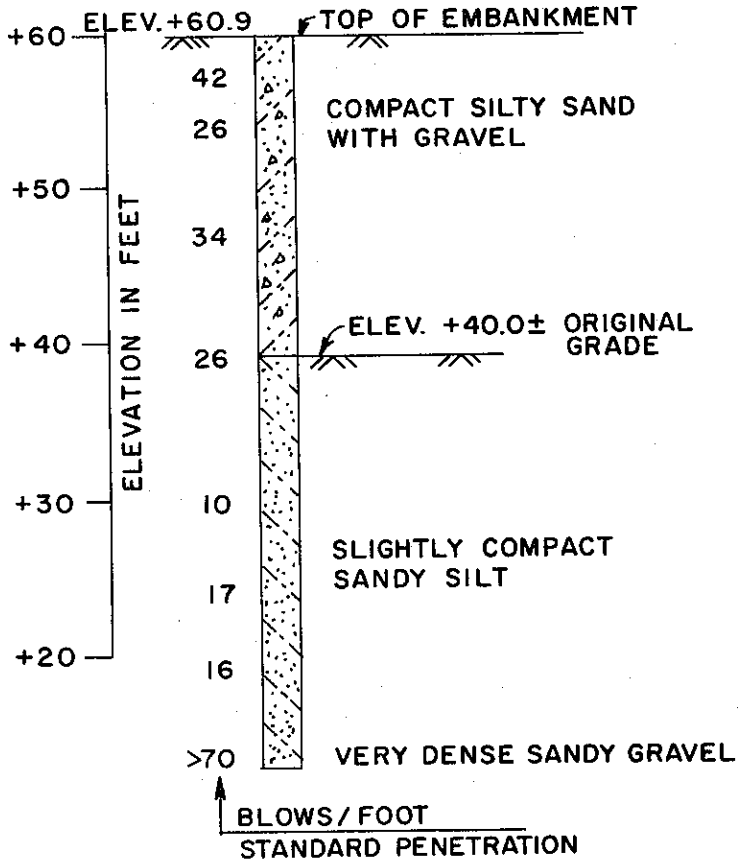
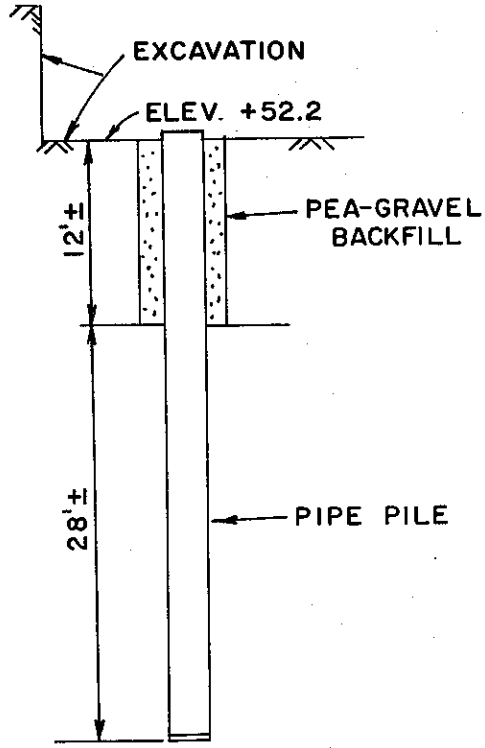


(a) PLAN OF TEST PILES



(b) BORING LOG



(c) TEST PILE

FIGURE 3

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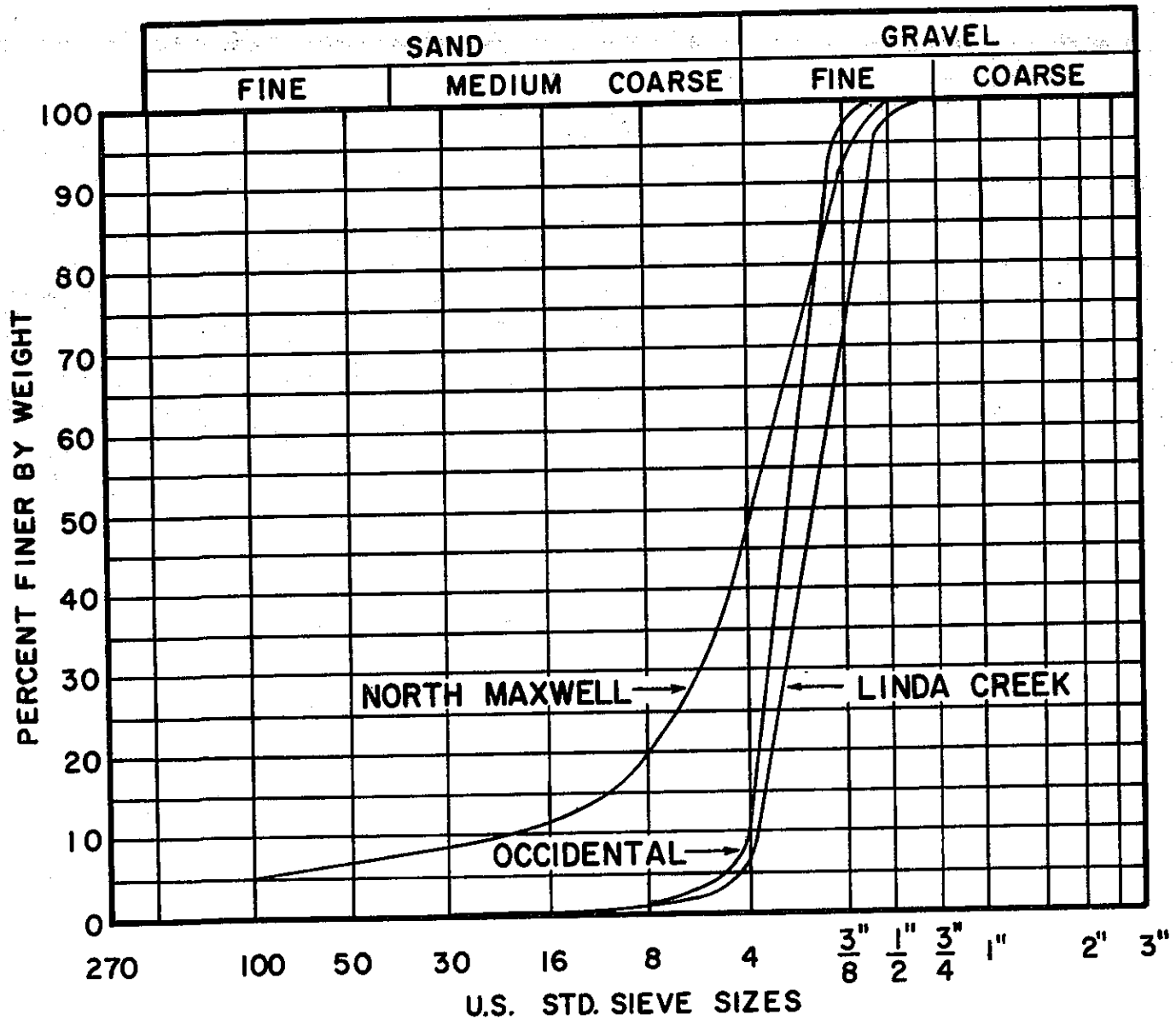
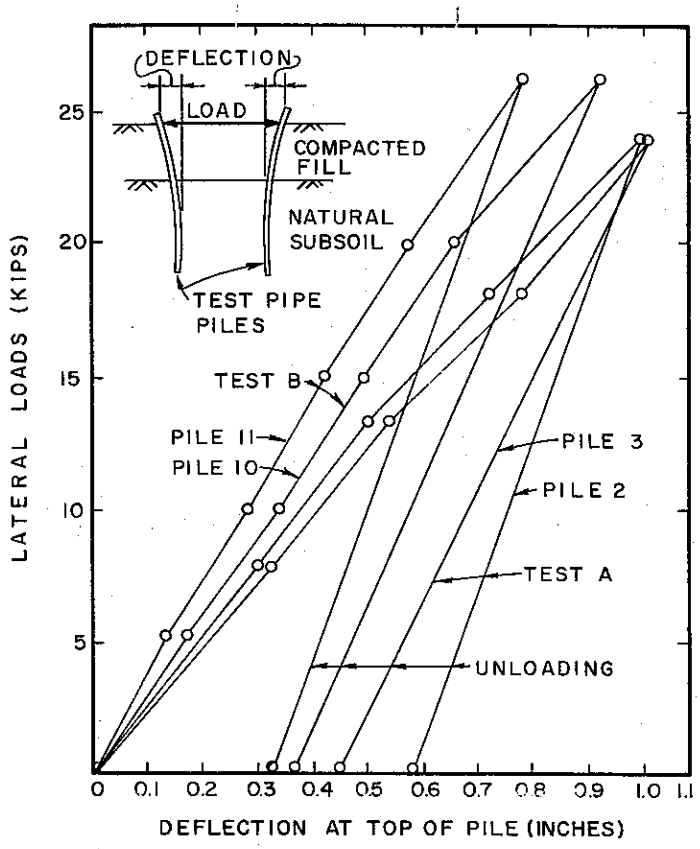
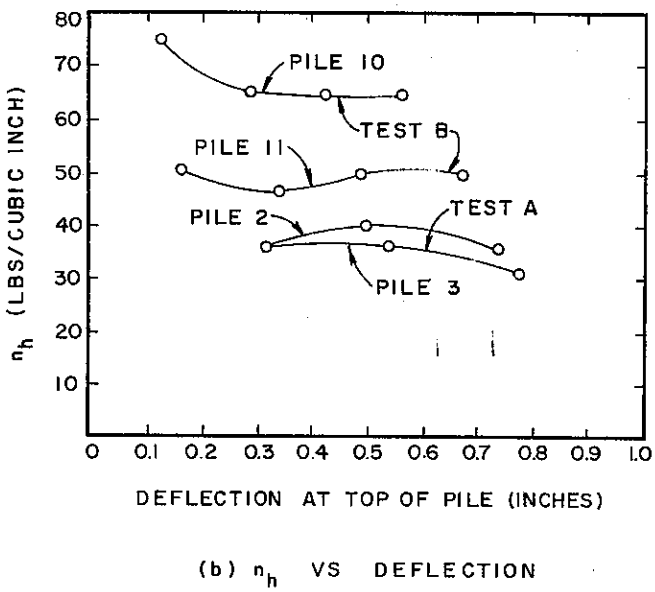


FIGURE 4



(a) LOAD VS DEFLECTION



(b) n_h VS DEFLECTION

FIGURE 5

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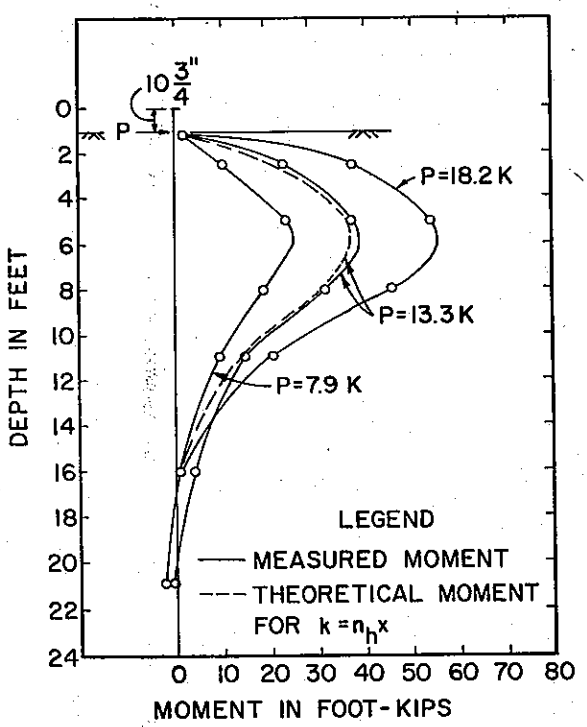
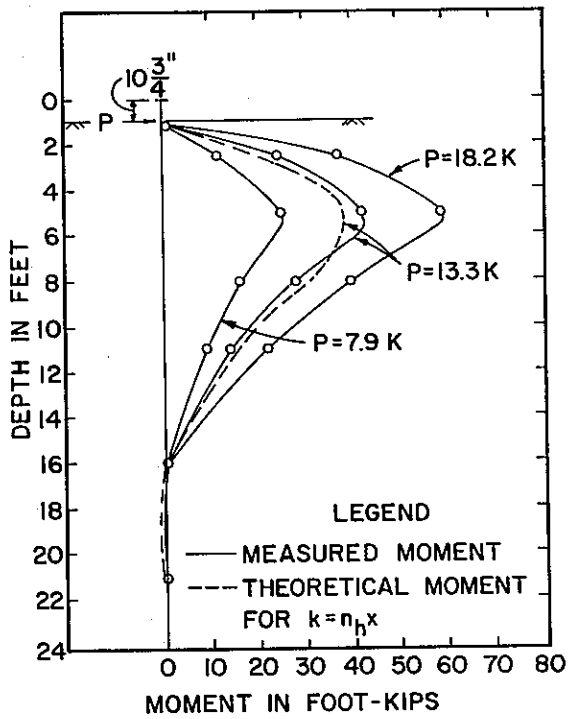
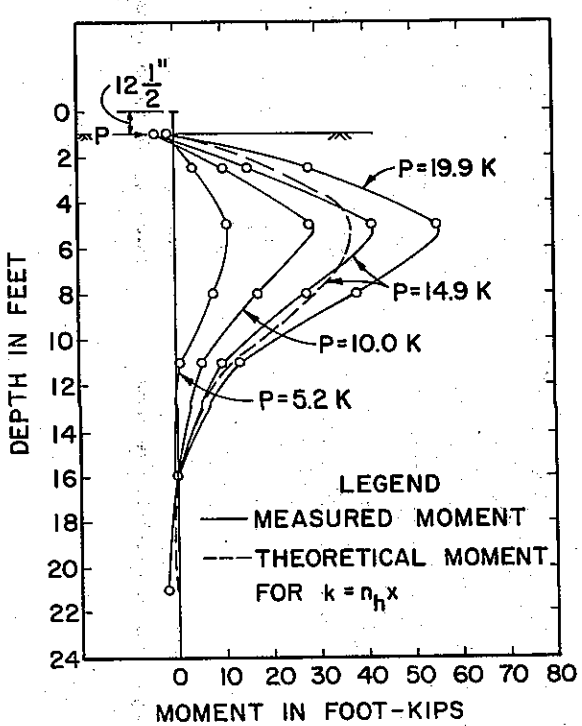


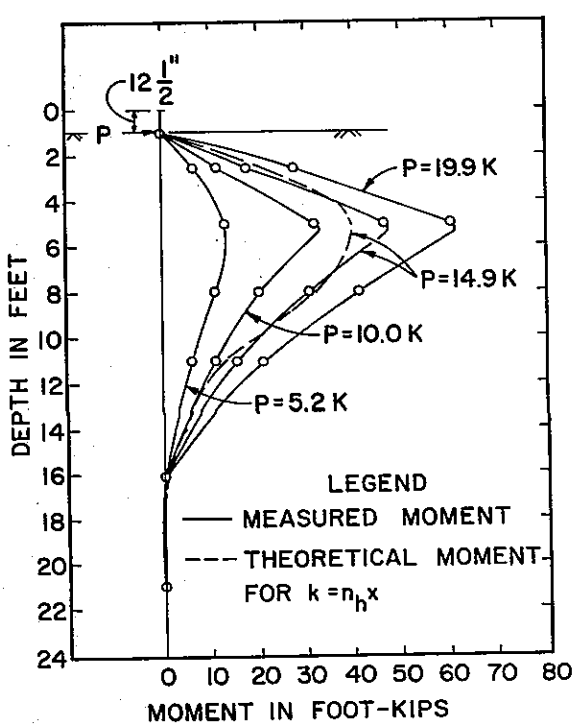
(a) TEST A, PILE 2



(b) TEST A, PILE 3



(c) TEST B, PILE 10



(d) TEST B, PILE 11

NOTE: TEST PILES 2, 3, 10 & 11 ARE $10\frac{3}{4}$" O.D. STEEL PIPE FILLED WITH REINFORCED CONCRETE

FIGURE 6

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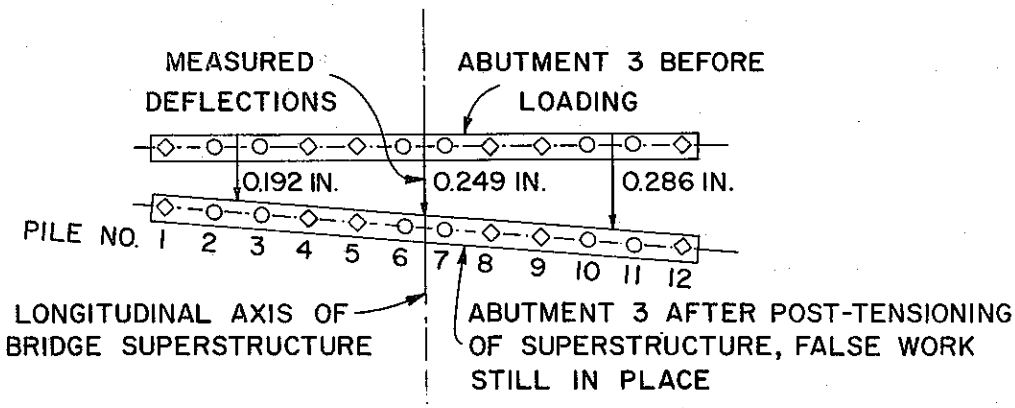
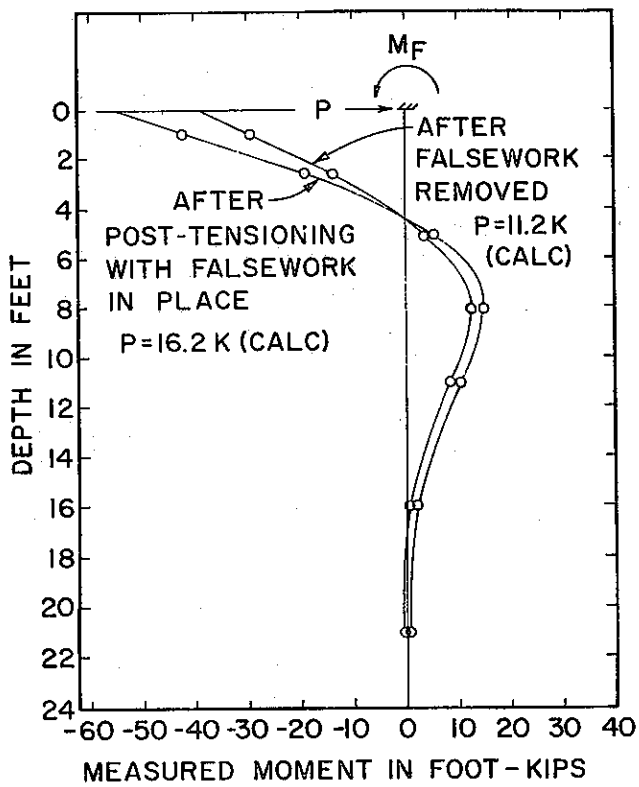
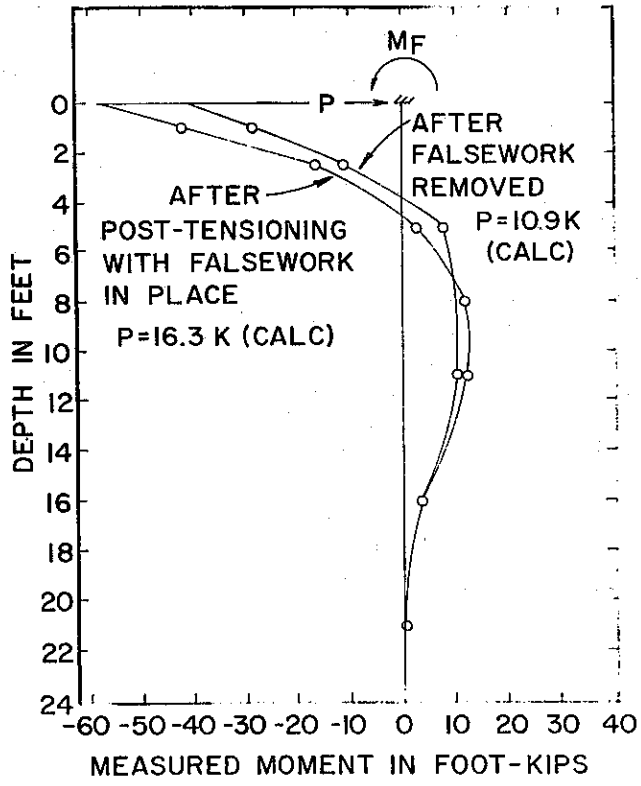


(a) PLAN OF LATERAL DEFLECTIONS - TEST C



(b) TEST C, PILE 6



(c) TEST C, PILE 7

FIGURE 7

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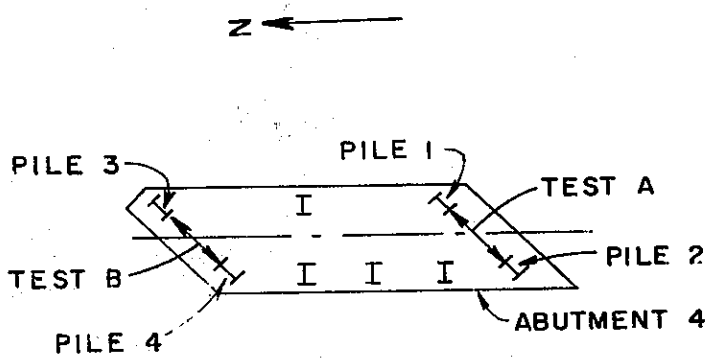
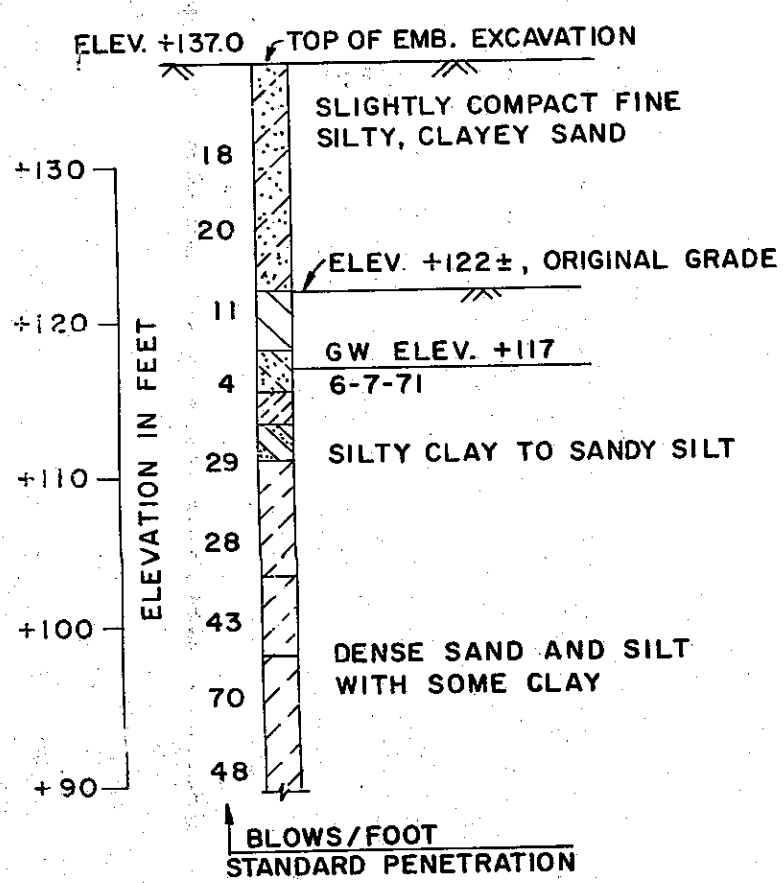
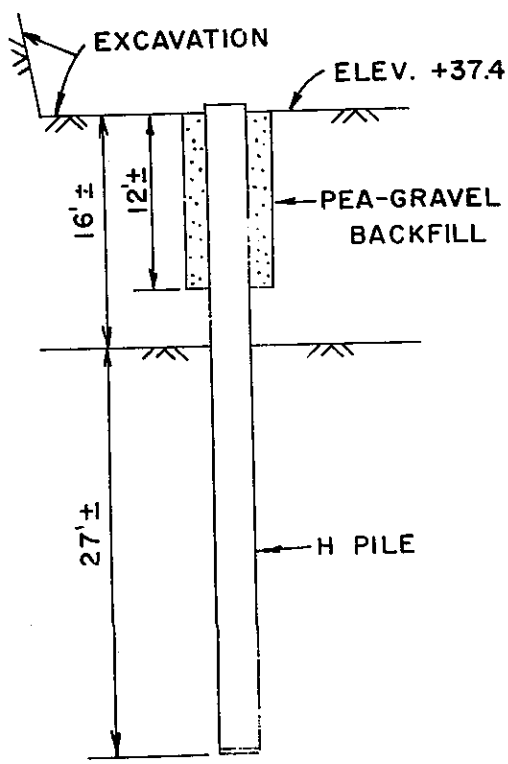


(a) PLAN OF TEST PILES



(b) BORING LOG



(c) TEST PILE

FIGURE 8

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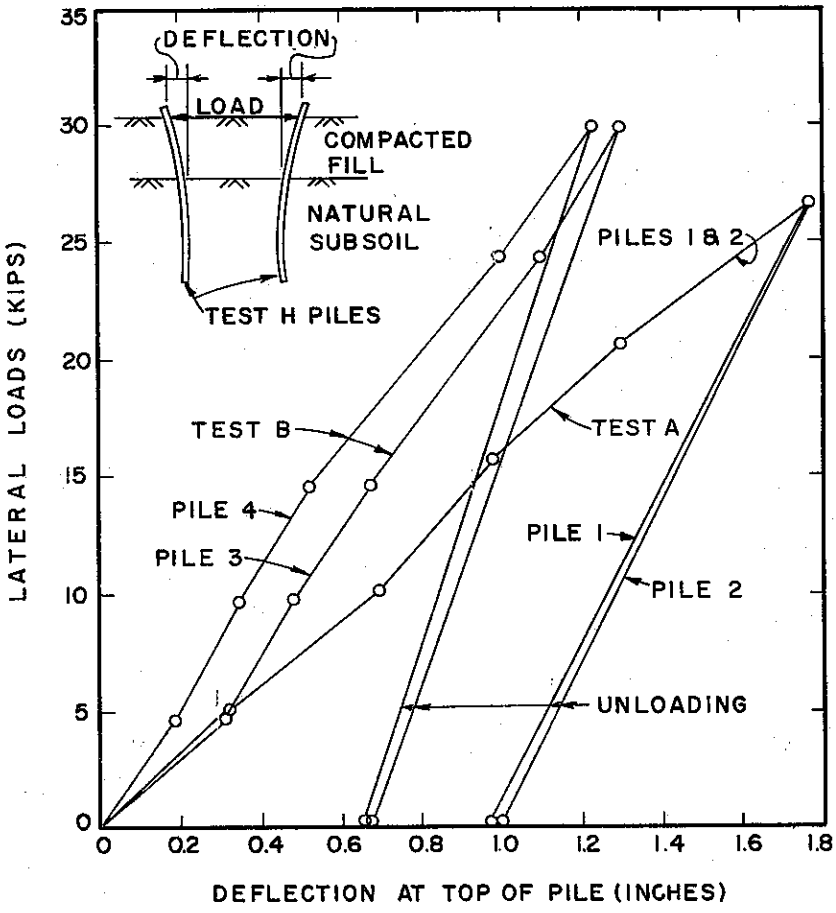
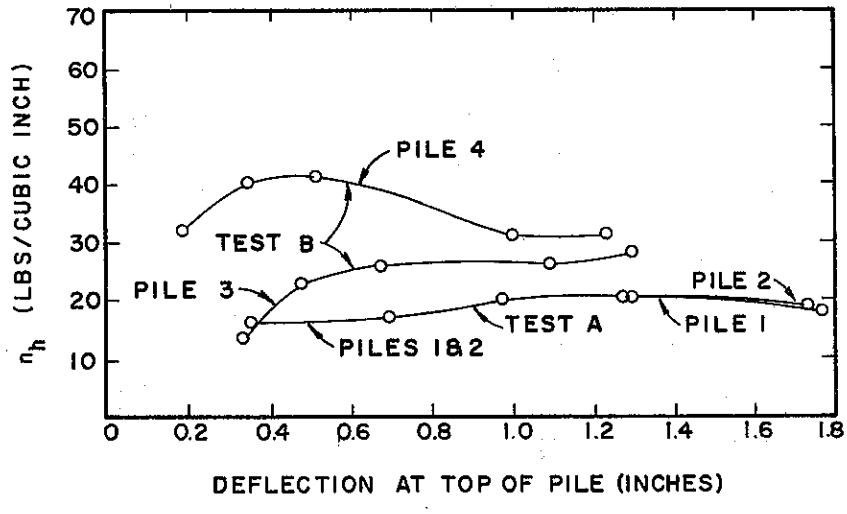


(a) LOAD VS DEFLECTION



(b) n_h VS DEFLECTION

FIGURE 9

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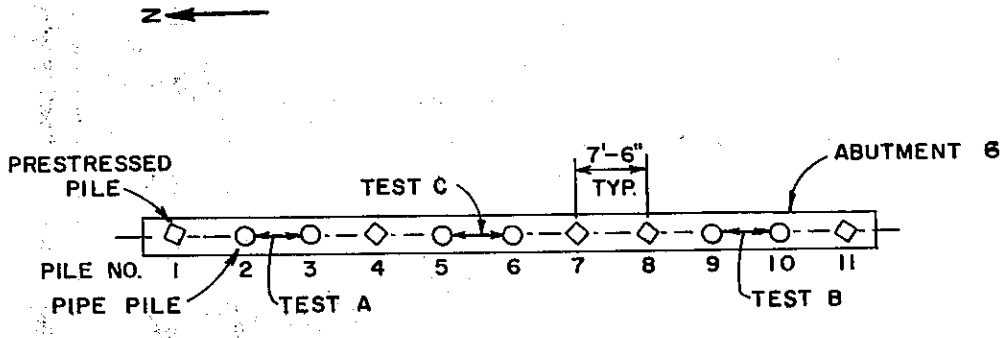
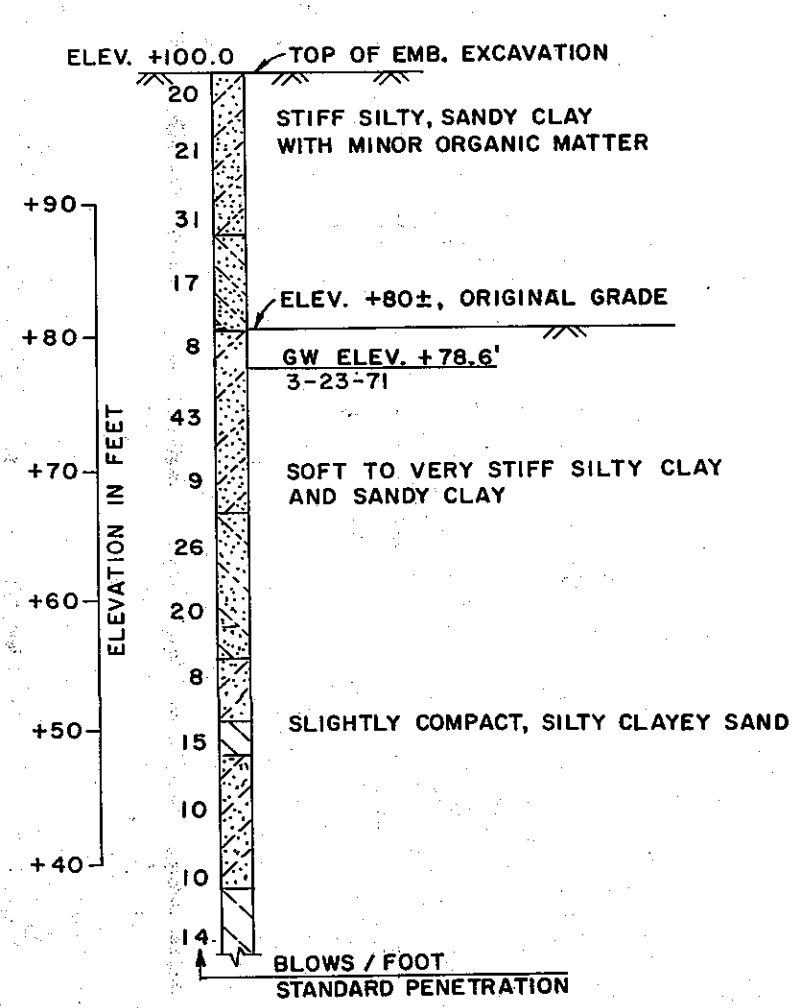


(a) PLAN OF TEST PILES



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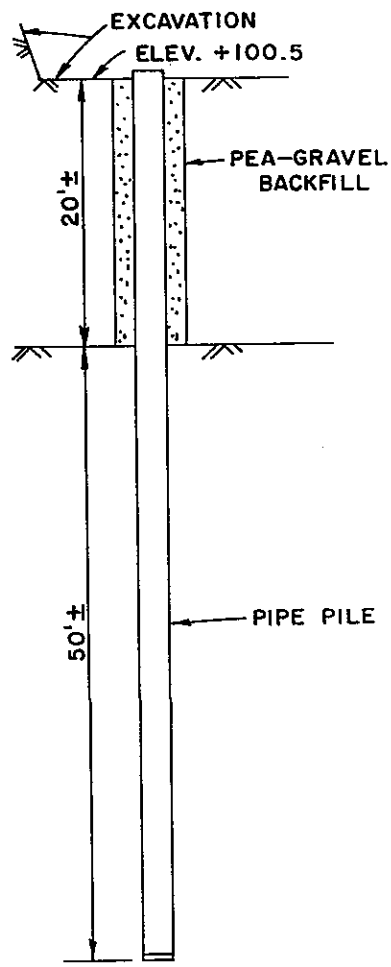

(c) TEST PILE

FIGURE 10

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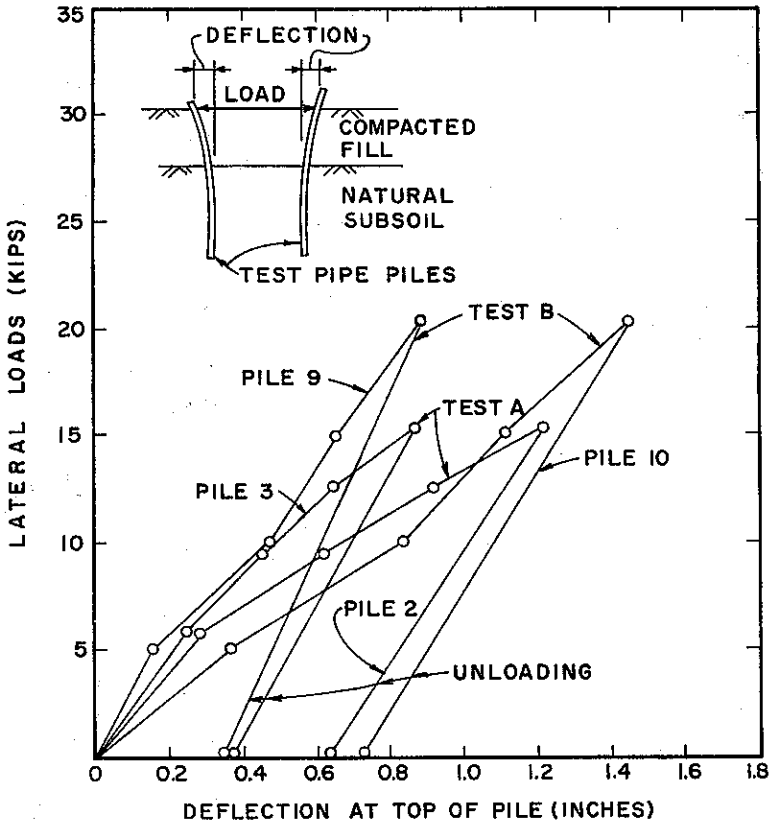
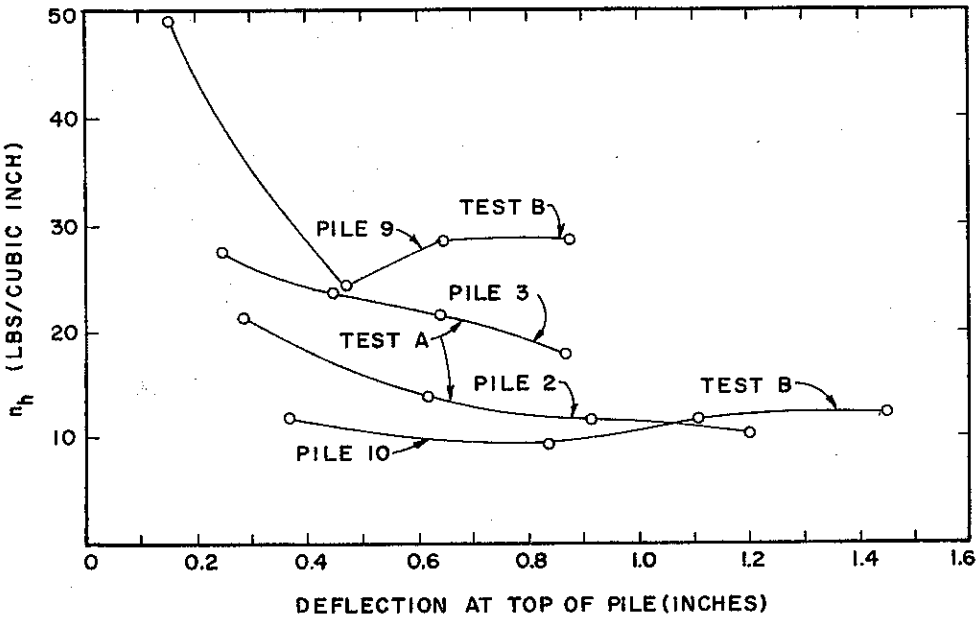


(a) LOAD VS DEFLECTION



(b) n_h VS DEFLECTION

FIGURE 11

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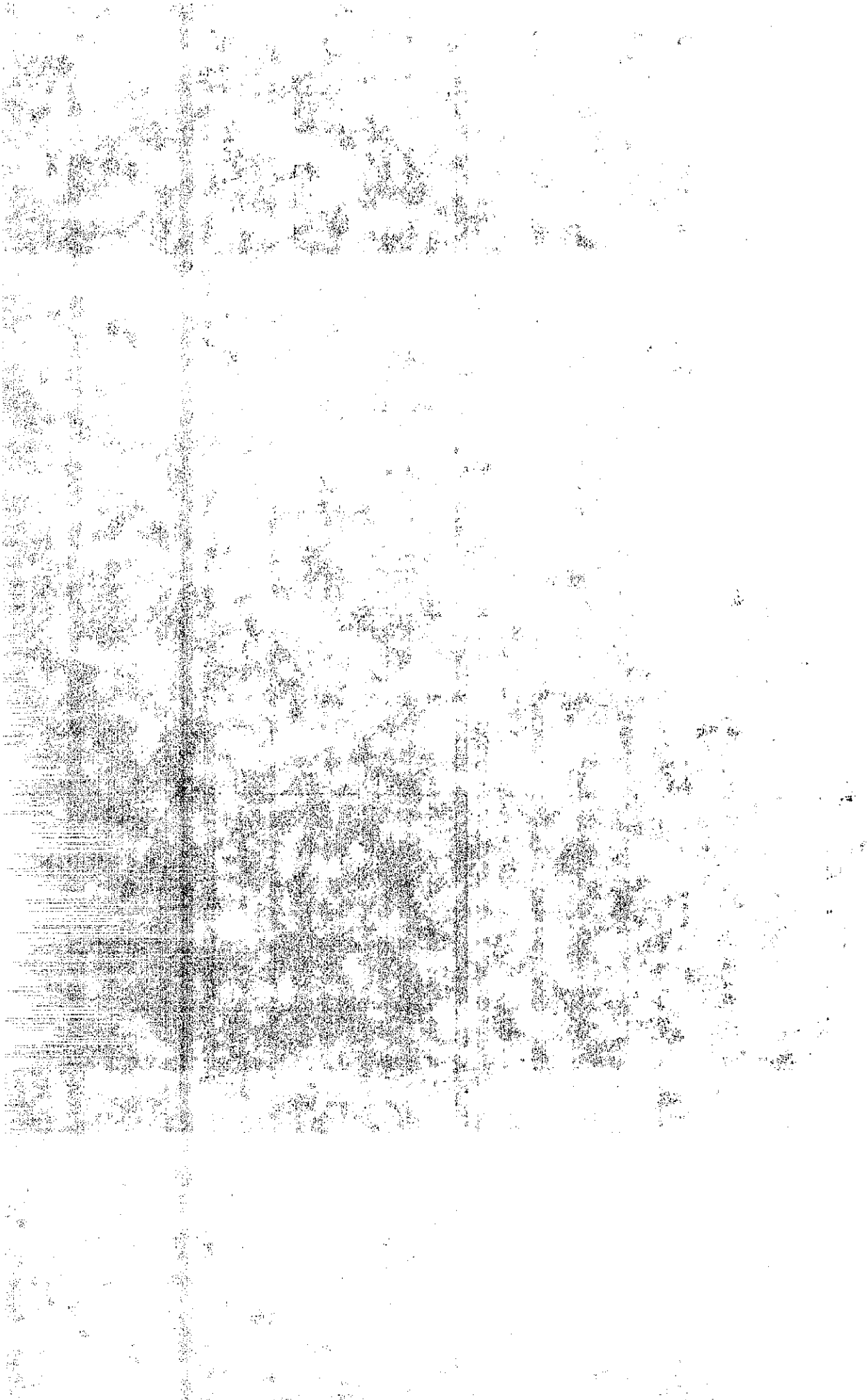

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