A STUDY OF BEARING CAPACITY OF PILE FOUNDATION

by 45

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INTRODUCTION

In the design of pile foundation the most important consideration is the evaluation of the bearing capacity of piles. Once the bearing capacity of a single pile is determined, either by dynamic formula or by static loading test, the engineer can then decide the number and length of piles needed for the foundation of structure and the effects of group reaction.

The evaluation of bearing capacity based on dynamic formulas has an advantage of economy both in time and in cost, but it has its disadvantage of giving unreliable result in most soil except in the non-cohesive sands and gravels. In these materials, the results from the formulas usually do not vary greatly from the results of full-size static loading test.

In the case of driving piles in plastic materials, such as soft clay or fine-grained silt, the relationship between the temporary resistance to driving and the permanent resistance to the working load on the pile varies greatly.

An exact criteria for the choice of dynamic formulas or static load testing is difficult to determine except by an understanding of soil characteristics and the different reactions of the pile under load.

PURPOSE OF THE STUDY

The purpose of this report was to investigate the bearing capacity of friction pile based on comparisons of results obtained from dynamic pile driving formulas and static pile loading tests.

SCOPE OF THE STUDY

The scope of the study included:

(a) A thorough research of the literature including a review of pile driving formulas and a study of the development and derivation of these formulas.

(b) A comparison of the results obtained from several selected formulas with assumed data to show variations of results.

(c) A general discussion of the static loading test.

(d) A suggestion for the evaluation of Q_f vs. Q_p by model pile tests.

(e) The principal consideration directed toward friction piles.

REVIEW OF THE LITERATURE

DYNAMIC FORMULAS

The earliest work on the bearing capacity of pile foundations indicated that the resistance of soil against the rapid penetration of a pile under the impact of the falling ram of the pile driver is theoretically proportional to the bearing capacity of the pile.

In about 1820, based on this relationship, Eytelwein proposed the first practical dynamic formula for pile driving:

$$Q = \frac{W_h \cdot H}{S(1+W_h/W_p)} \tag{1}$$

in which ; Q = the bearing capacity of the pile,

S = average penetration per blow,

W_h = the weight of hammer,

W_p = the weight of pile,

H = height of fall of the hammer.

In 1851, Sander developed his formula based on the same concept that the energy from the falling hammer equals the work accomplished or

$$E = \text{Work} = Q \cdot S$$

E is the energy from the falling hammer.

Since E = (weight of hammer)$\cdot$(Height of hammer falling)

$$= W_h \cdot H$$

we have thus the Sander's formula for dynamic driving of pile as:

$$Q = \frac{W_h \cdot H}{S} \tag{2}$$

The formula from Eytelwein and from Sander are used for the ideal condition in which there is no loss of energy during driving. In actuality an amount of energy is lost due to

1) Elastic deformation of pile,
2) Plastic deformation of pile,
3) Elastic deformation of soil,
4) Plastic deformation of soil,
5) The efficiency of the driving equipment.

In considering the loss of energy in driving of pile, Sander modified his formula to

$$Q = \frac{W_h \cdot H}{S + C} \tag{3}$$

C is a constant from experimental data for energy loss.

In 1859, Redtenbacker derived a formula based on Newton's Impact Theory and set some assumptions which neglected some of the driving energy loss. His formula is

$$Q = \frac{S\ A\ E}{L}\left[\sqrt{1 + \frac{2LW_h^2H}{S\cdot A\cdot E\cdot (W_p+W_h)}} - 1\right] \tag{4}$$

In 1888, Wellington determined the value of C for Sander's formula by gathering experimental field data determining that C = 1.0 for a simple drop hammer and C = 0.1 for single acting hammer. In the formula, S and H are in terms of inches.

By comparing the results found by his formula with the results from actual static loading test, Wellington proposed the most commonly used "Engineering News Formula" by setting a safety factor 1/6 in (3), which gives:

$$Q = (1/6) \cdot \frac{W_h \cdot H}{S+1} \tag{5}$$

for a drop hammer, and

$$Q = (1/6) \cdot \frac{W_h \cdot H}{S+0.1} \tag{5a}$$

for single acting hammer.

In the early 1900's, this Engineering News Formula was widely used in the United States and in Europe.

In 1930, A. Hiley published through the Institute of Civil Engineering a pile driving formula which considered every loss possible and is known as the "Complete" pile driving formula

$$Q = \frac{(S+S_f)AE}{CL} \left[\sqrt{1 + \frac{2\ C\ L\ e_1\ W_h\ H}{(S+S_f)^2 AE} \cdot \frac{W_h + n^2 W_p}{W_h + W_p}} - 1 \right] \tag{6}$$

He simplified it by assuming C = 1

$$Q = \frac{(S+S_f)AE}{L} \left[\sqrt{1 + \frac{2\ L\ e_1\ W_h\ H}{(S+S_f)^2 AE} \cdot \frac{W + n^2\ W_p}{W_h + W_p}} - 1 \right] \quad (6a)$$

Referring to equation (4), it is obvious that Redtenbacker's formula is the same as Hiley's by the simplifying assumptions that

$S_f = 0,$

$e_1 = 1,$ and

$n = 0.$

DERIVATION OF THE GENERAL FORMULA

The simplest form of the driving formulas is the original Hiley's dynamic formula which is based on:

Driving energy = Work of pile penetration + energy losses

We know that the energy supplied by the hammer is

$$E_0 = W_h \cdot H \qquad (7)$$

W_h = weight of hammer

H = falling height of hammer.

It is reasonable to expect some energy lost by the falling of hammer. If e_1 is the efficiency of the hammer fall then the energy reaching the pile is

$$E_1 = e_1 \cdot E_0 = e_1 \cdot W_h \cdot H = e_1 \cdot W_h \cdot \frac{V_h^2}{2g} \qquad (8)$$

V_h = the velocity of hammer prior to striking.

According to Newton's impact relationship, the efficiency of impact is:

$$e_2 = \frac{\frac{W_h}{2g} \cdot v_h^2 + \frac{W_p}{2g} \cdot v_p^2}{\frac{W_h}{2g} \cdot V_h^2 + \frac{W_p}{2g} \cdot V_p^2} \qquad (9)$$

for which

V_p = velocity of pile prior to being struck,

v_p = velocity of pile after being struck,

v_h = velocity of hammer after striking pile.

The denominator is the energy before striking and the numerator is the energy after striking.

The impulses of the hammer and pile are equal during contact. The momentum change, which is equal to the impulse, is the product of mass and velocity change. Assuming both the hammer and the pile are free, massive bodies (which is not strictly true for the pile) we have

$$\frac{W_h}{g}(V_h - v_h) = \frac{W_p}{g}(V_p - v_p) \qquad (10)$$

Another important coefficient that is needed is "Newton's coefficient of elastic restitution", defined as

$$n = \frac{v_p - v_h}{V_h - v_p} \qquad (11)$$

From experimental data, constant values for n have been discovered which vary according to the material of the piles but are usually in the range of 0.25 to 0.55. Reference for the n value are shown in Appendix (B).

From Eq. (10) and (11), since $V_p = 0$, we have

$$W_h(V_h - v_p) = W_p \cdot (-v_p) \qquad (10a)$$

and

$$nV_h - nv_p + v_h = 0 \qquad (11a)$$

From these two equations, v_h and v_p can be expressed in terms of n and V_h,

$$v_p = \frac{(1 + n)\ W_h}{n\ W_h - W_p} \cdot V_h$$

$$v_h = \frac{W_h + W_p}{nW_h - W_p} \cdot n \cdot V_h \qquad (12)$$

Substituting (12) into (9),

$$e_2 = \frac{W_h\ v_h^2 + W_p\ v_p^2}{W_h \cdot V_h^2}$$

$$= \frac{W_h\ V_h^2\ n^2 (\frac{W_h + W_p}{nW_p - W_p})^2 + W_p V_h^2 (\frac{(1+n) W_h}{nW_h - W_p})^2}{W_h \cdot V_h^2}$$

which can be simplified to:

$$e_2 = \frac{W_h + n^2\ W_p}{W_h + W_p} \qquad (13)$$

By the definition of e_2, the energy available after impact with the pile is

$$E_2 = e_2\ E_1 = e_1\ e_2\ E_0 = e_1\ E_0\ (\frac{W_h + n^2\ W_p}{W_h + W_p})$$

$$= e_1\ W_h\ H\ (\frac{W_h + n^2\ W_p}{W_h + W_p})$$

The amount of energy E_2 is used up in the useful work of forcing the pile into the ground $(Q \cdot S)$, in losses due to crushing of material at the pile head $(Q \cdot S_f)$, and in losses due to the elastic compression of the soil-pile system $(Q \cdot S_e)$. Thus

$$E_2 = Q(S + S_f + S_e) \tag{15}$$

In considering the elastic compression of pile, there are two different types of compression forces to balance the loading from the top of the pile. The first type is the end bearing force and the other is the side frictional force. For frictional pile, the end bearing is small compared to the side frictional force and can be neglected. When we evaluate the elastic compression of friction pile the variance of frictional stress is based on the assumption that static earth pressure is increasing with depth and the magnitude of frictional stress is in proportion to the normal pressure on the pile surface which is actually static lateral earth pressure from the surrounding soil.

If we let σ be the maximum frictional stress which reasonably exists at point B. For every element of the pile with length dx will have a compression $d(S_e)$, while

$$d(S_e) = (L - x)\left(\frac{\sigma + \frac{x}{L}\sigma}{2}\right) \frac{\pi d \cdot dx}{A\ E}$$

$$(S_e)1 = \int_0^L (L - x)\left(\frac{\sigma + \frac{x}{L}\sigma}{2}\right) \frac{\pi \cdot d \cdot dx}{A\ E}$$

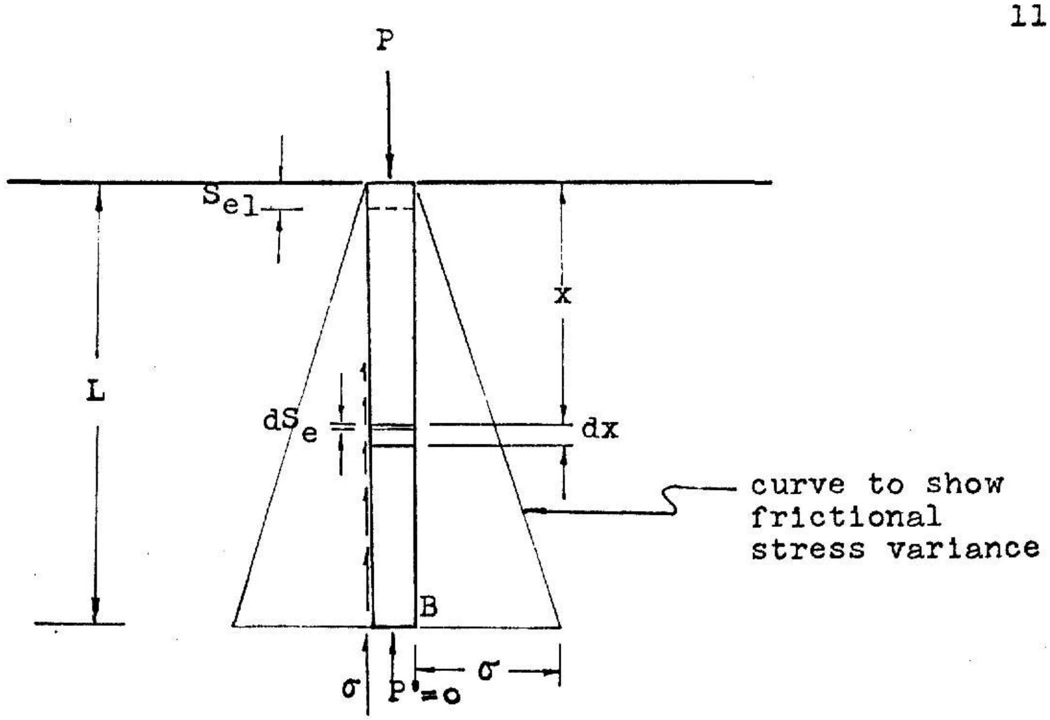

Fig. 4 Friction pile under load.

$$(S_e)_1 = \frac{\pi\ d\ \sigma}{2 \cdot AEL} \int_0^L (L - x)(L + x)\ dx$$

$$= \frac{\pi\ d\ \sigma}{2AEL} \int_0^L (L^2 - x^2)\ dx$$

$$= \frac{\pi\ d\ \sigma}{2AEL} \left[L^2 x - \frac{x^3}{3}\right]_0^L$$

$$= \frac{\pi\ d\ \sigma}{2AEL} \cdot \frac{2 \cdot L^3}{3}$$

$$= (2/3)\ \frac{P \cdot L}{A \cdot E} \qquad P = \frac{\pi\ d\ L}{2}$$

More elementary than the above case, is an end bearing pile loaded by axial loading P with a reaction force P at the pile tip and no frictional force along the pile surface. When we calculate the elastic compression of pile for this case:

$$(S_e)_1 = \frac{P \cdot L}{A \cdot E} = (1)\frac{P \cdot L}{A \cdot E}$$

In actuality, there are no pile which takes loading only by side friction or by end bearing but most are a combination of both. If we let:

$$(S_e) = C \frac{P \cdot L}{A \cdot E} \quad (16)$$

It is obvious that C should be between 0.67 to 1.00, for absolute end bearing pile C = 1.00 and for absolute friction pile C = 0.67.

Combining Eq. (14), (15) and (16) to obtain Q_u:

$$Q_u = \frac{e_1 \cdot W_h \cdot H}{S + S_f + \frac{C \cdot Q_u \cdot L}{A\ E}} \cdot \frac{W_h + n^2\ W_p}{W_h + W_p}$$

Rearranged as:

$$Q_u = \frac{(S + S_f) \cdot A \cdot E}{C\ L} \left[\sqrt{1 + \frac{2CLe_1 W_h H}{(S+S_f)^2 AE} \quad \frac{W_h + n^2 W_p}{W_h + W_p}} - 1 \right]$$

This is the basic, general form of dynamic formula based on Newton's theory of impaction. Many simplified formulas were obtained by modifying this formula with various assumptions. The results from the original formula and the simplified formulas vary because of these assumptions. A numerical test for several formulas is given in the next section. A discussion concerning those assumptions is also included in the next section.

COMPARISON OF RESULTS USING DIFFERENT DYNAMIC FORMULAS

Three dynamic formulas (1) the general formula from Hiley, (2) that of Redtenbacker and (3) the Engineering News Formula, are considered in this comparison by substituting comparable numerical data into each formula.

Resulting curves of "bearing capacity Q" vs. "the average penetration per blow S" are plotted and the assumptions are discussed based on the results of calculated ultimate bearing capacity obtained from the three different formulas.

To show how the results differed for short, medium, and long piles, three different size of precast concrete piles were used; 50 ft with 10 inch diameter, 70 ft with 12 inch diameter, and 100 ft with 14 inch diameter. Three drop hammer of different weight were used for each pile; 2,500 lb for 50 ft pile, 3,500 lb for 70 ft pile, and 6,000 lb for 100 ft pile.

A computer program was prepared for these tests and three different sets of input data were made for the three piles. The data used are listed below:

E = modulus of elasticity = $3 \quad 10^6$ psi.

e_1 = efficiency of hammer fall = 0.75 for drop hammer

H = height of hammer drop = 15 ft=180 inch

n = Newton's coefficient of elastic restitution
= 0.4 for concrete pile.

C = constant of the elastic deformation of pile under load

= 0.67 for friction pile.

S_f = non-elastic crushing of pile per blow

= (0.1) S assumed

S = average penetration of pile per blow

= 0.1, 0.2, ... 3.9, 4.0.

L = length of the piles: L_1 = 50 ft = 600 inch,

L_2 = 70 ft = 840 inch,

L_3 = 100 ft = 1200 inch.

A = average cross sectional area of piles:

A_1 = 78.5 sq in.,

A_2 = 113.0 sq in.,

A_3 = 158.0 sq in.

W_p = weight of piles: W_{p1} = 4,100 lb,

W_{p2} = 8,240 lb,

W_{p3} = 19,800 lb.

W_h = weight of hammer: W_{h1} = 2,500 lb,

W_{h2} = 3,500 lb,

W_{h3} = 6,000 lb.

The formulas used are:

1) General form of dynamic formula from Hiley,

$$Q_h = \frac{(S+S_f)\ AE}{CL}\left[\sqrt{1 + \frac{2Le_1W_h\ H}{(S+S_f)^2 AE} \cdot \frac{W_h\ n^2\ W_p}{W_h + W_p}} - 1\right]$$

2) Redtenbacker's formula,

$$Q_r = \frac{S\ A\ E}{L}\left[\sqrt{1 + \frac{2\ L\ W_h^2\ H}{S^2\ AE(W_h+W_p)}} - 1\right]$$

3) Engineering News Formula,

$$Q_e = (1/6)\ \frac{W_h \cdot H}{S + 1}$$

Three tables and charts for 50 ft, 70 ft, and 100 ft piles are made when the data are substituted into the above formulas. From the charts it is obvious that the length of piles didn't influence the shape of the curves greatly. For all the three formulas, the curves are almost straight lines and shown linear relationship between Q and S for very soft soil. That is, when the average penetrations of piles are more than 2.5 inch, it will be found that $Q = c_1\ S$ and c_1 is a constant.

When s decreases from S = 2.5 inch, the curves turn slightly right and have higher bearing capacity for decreasing S values. The curves are more curved when s is about 1.5" to 0.5".

The curve from the general formula and that from Redtenbacker's are about the same shape and the values of Q_h are always higher than the values of Q_r for all kinds of soil conditions (from very hard to very soft).

If we let $Q_h = X \cdot Q_r$, the curves shown X vs. s can be made as the chart on page 22.

For the shortest pile, the values of X vary from 1.43 when S = 0.5" to 1.29" when S = 4.0". The difference of the maximum tp minimum values of X is only 0.14 which is small when compared to the variance of S.

Table 1. Bearing capacities of pile for variance of average penetration when L= 50 ft.

C C COMPUTER PROGRAM FOR PILE FORMULAS

S	QH	QR	QE	X
.10	235.35	164.32	34.09	1.43
.20	208.89	147.81	31.25	1.41
.30	186.00	133.27	28.85	1.40
.40	166.36	120.53	26.79	1.38
.50	149.56	109.43	25.00	1.37
.60	135.20	99.77	23.44	1.36
.70	122.92	91.37	22.06	1.35
.80	112.37	84.04	20.83	1.34
.90	103.27	77.63	19.74	1.33
1.00	95.37	72.01	18.75	1.32
1.10	88.49	67.06	17.86	1.32
1.20	82.45	62.68	17.05	1.32
1.30	77.12	58.79	16.30	1.31
1.40	72.40	55.31	15.63	1.31
1.50	68.18	52.19	15.00	1.31
1.60	64.41	49.38	14.42	1.30
1.70	61.01	46.84	13.89	1.30
1.80	57.93	44.54	13.39	1.30
1.90	55.14	42.44	12.93	1.30
2.00	52.60	40.52	12.50	1.30
2.10	50.27	38.76	12.10	1.30
2.20	48.13	37.14	11.72	1.30
2.30	46.17	35.65	11.36	1.30
2.40	44.35	34.26	11.03	1.29
2.50	42.67	32.98	10.71	1.29
2.60	41.10	31.79	10.42	1.29
2.70	39.65	30.68	10.14	1.29
2.80	38.29	29.64	9.87	1.29
2.90	37.02	28.67	9.62	1.29
3.00	35.83	27.75	9.38	1.29
3.10	34.72	26.90	9.15	1.29
3.20	33.67	26.09	8.93	1.29
3.30	32.68	25.33	8.72	1.29
3.40	31.74	24.61	8.52	1.29
3.50	30.86	23.93	8.33	1.29
3.60	30.02	23.29	8.15	1.29
3.70	29.23	22.68	7.98	1.29
3.80	28.48	22.10	7.81	1.29
3.90	27.77	21.55	7.65	1.29
4.00	27.09	21.03	7.50	1.29

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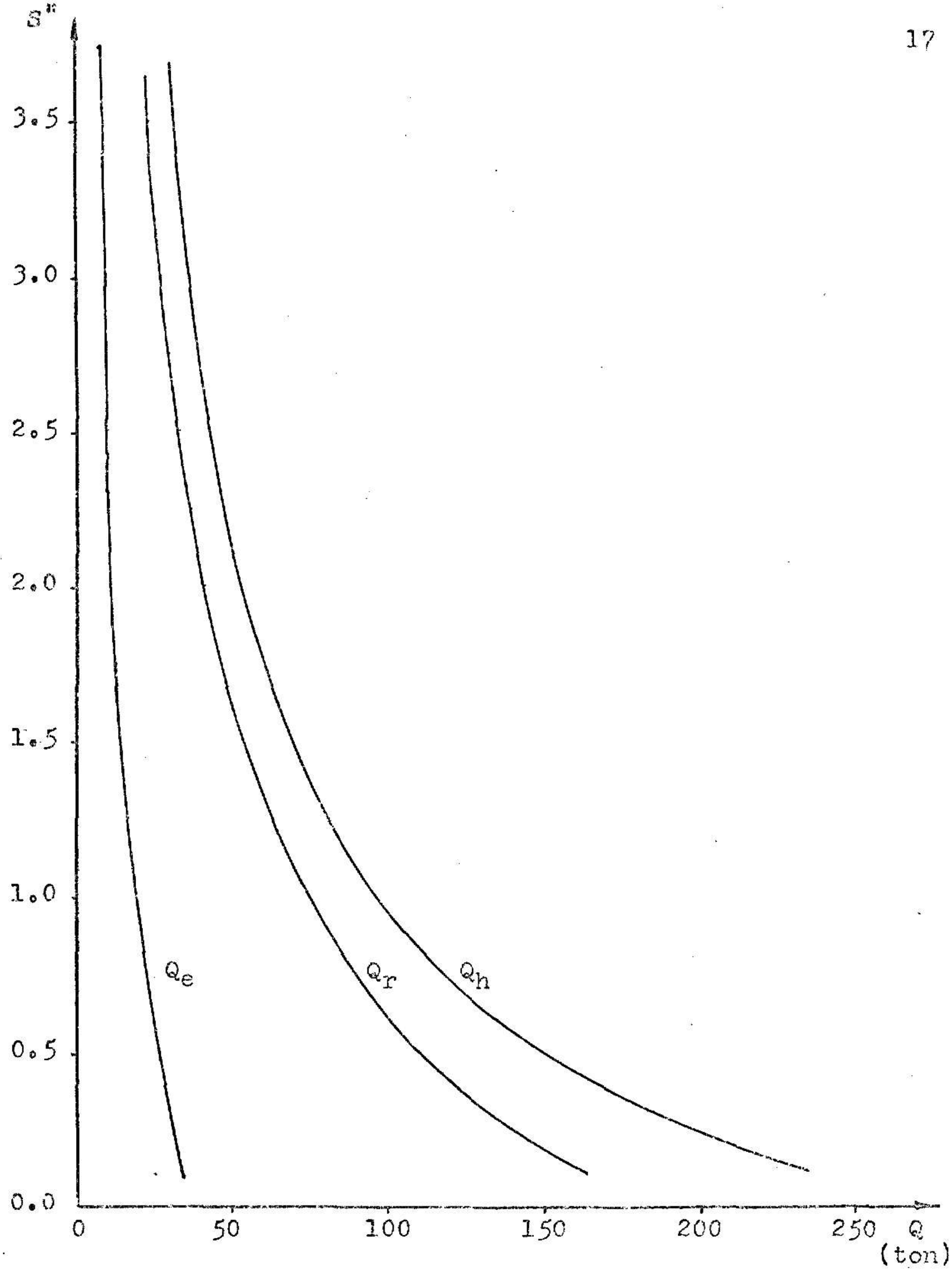

Fig. 5. Curves of bearing capacity (Q) vs. average penetration (S) when L= 50 ft.

Table 2. Bearing capacities of pile for variance of average penetration when L= 70 ft.

C C COMPUTER PROGRAM FOR PILE FORMULAS

S	QH	QR	QE	X
.10	263.97	175.54	47.73	1.50
.20	236.33	158.46	43.75	1.49
.30	212.14	143.34	40.38	1.48
.40	191.11	130.03	37.50	1.47
.50	172.90	118.38	35.00	1.46
.60	157.15	108.18	32.81	1.45
.70	143.53	99.27	30.88	1.45
.80	131.72	91.47	29.17	1.44
.90	121.45	84.63	27.63	1.44
1.00	112.47	78.60	26.25	1.43
1.10	104.59	73.28	25.00	1.43
1.20	97.65	68.55	23.86	1.42
1.30	91.49	64.35	22.83	1.42
1.40	86.00	60.58	21.88	1.42
1.50	81.09	57.20	21.00	1.42
1.60	76.68	54.15	20.19	1.42
1.70	72.70	51.39	19.44	1.41
1.80	69.09	48.88	18.75	1.41
1.90	65.80	46.59	18.10	1.41
2.00	62.81	44.50	17.50	1.41
2.10	60.06	42.58	16.94	1.41
2.20	57.53	40.81	16.41	1.41
2.30	55.20	39.18	15.91	1.41
2.40	53.05	37.66	15.44	1.41
2.50	51.05	36.26	15.00	1.41
2.60	49.20	34.95	14.58	1.41
2.70	47.47	33.74	14.19	1.41
2.80	45.85	32.60	13.82	1.41
2.90	44.34	31.53	13.46	1.41
3.00	42.93	30.53	13.13	1.41
3.10	41.60	29.59	12.80	1.41
3.20	40.35	28.71	12.50	1.41
3.30	39.17	27.87	12.21	1.41
3.40	38.05	27.09	11.93	1.40
3.50	37.00	26.34	11.67	1.40
3.60	36.00	25.63	11.41	1.40
3.70	35.06	24.96	11.17	1.40
3.80	34.16	24.33	10.94	1.40
3.90	33.30	23.72	10.71	1.40
4.00	32.49	23.15	10.50	1.40

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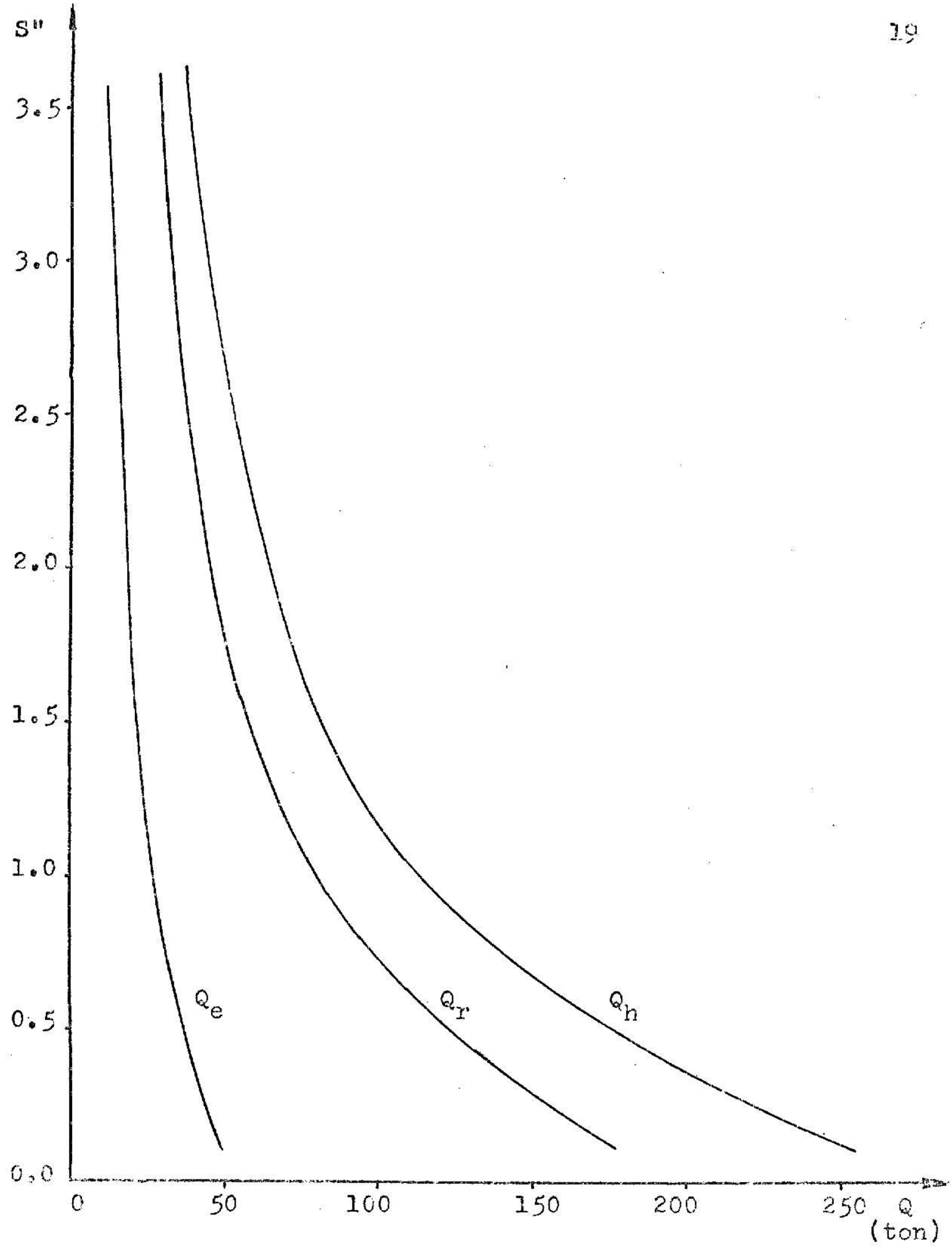

Fig. 6. Curves of bearing capacity (Q) vs. average penetration (S) when L = 70 ft.

Table 3. Bearing capacities of pile for variance average penetration when L = 100 ft.

C C COMPUTER PROGRAM FOR PILE FORMULAS

S	QH	QR	QE	X
.10	324.91	203.85	81.82	1.59
.20	296.87	186.70	75.00	1.59
.30	271.64	171.22	69.23	1.59
.40	249.06	157.32	64.29	1.58
.50	228.92	144.88	60.00	1.58
.60	211.02	133.78	56.25	1.58
.70	195.11	123.89	52.94	1.57
.80	180.97	115.07	50.00	1.57
.90	168.39	107.21	47.37	1.57
1.00	157.18	100.18	45.00	1.57
1.10	147.16	93.88	42.86	1.57
1.20	138.19	88.23	40.91	1.57
1.30	130.13	83.14	39.13	1.57
1.40	122.86	78.54	37.50	1.56
1.50	116.28	74.38	36.00	1.56
1.60	110.32	70.60	34.62	1.56
1.70	104.89	67.16	33.33	1.56
1.80	99.93	64.01	32.14	1.56
1.90	95.39	61.12	31.03	1.56
2.00	91.22	58.46	30.00	1.56
2.10	87.38	56.02	29.03	1.56
2.20	83.83	53.76	28.13	1.56
2.30	80.55	51.66	27.27	1.56
2.40	77.50	49.72	26.47	1.56
2.50	74.67	47.91	25.71	1.56
2.60	72.03	46.22	25.00	1.56
2.70	69.56	44.64	24.32	1.56
2.80	67.25	43.17	23.68	1.56
2.90	65.08	41.78	23.08	1.56
3.00	63.05	40.48	22.50	1.56
3.10	61.13	39.25	21.95	1.56
3.20	59.33	38.10	21.43	1.56
3.30	57.62	37.00	20.93	1.56
3.40	56.01	35.97	20.45	1.56
3.50	54.48	34.99	20.00	1.56
3.60	53.04	34.07	19.57	1.56
3.70	51.66	33.19	19.15	1.56
3.80	50.36	32.35	18.75	1.56
3.90	49.12	31.55	18.37	1.56
4.00	47.93	30.79	18.00	1.56

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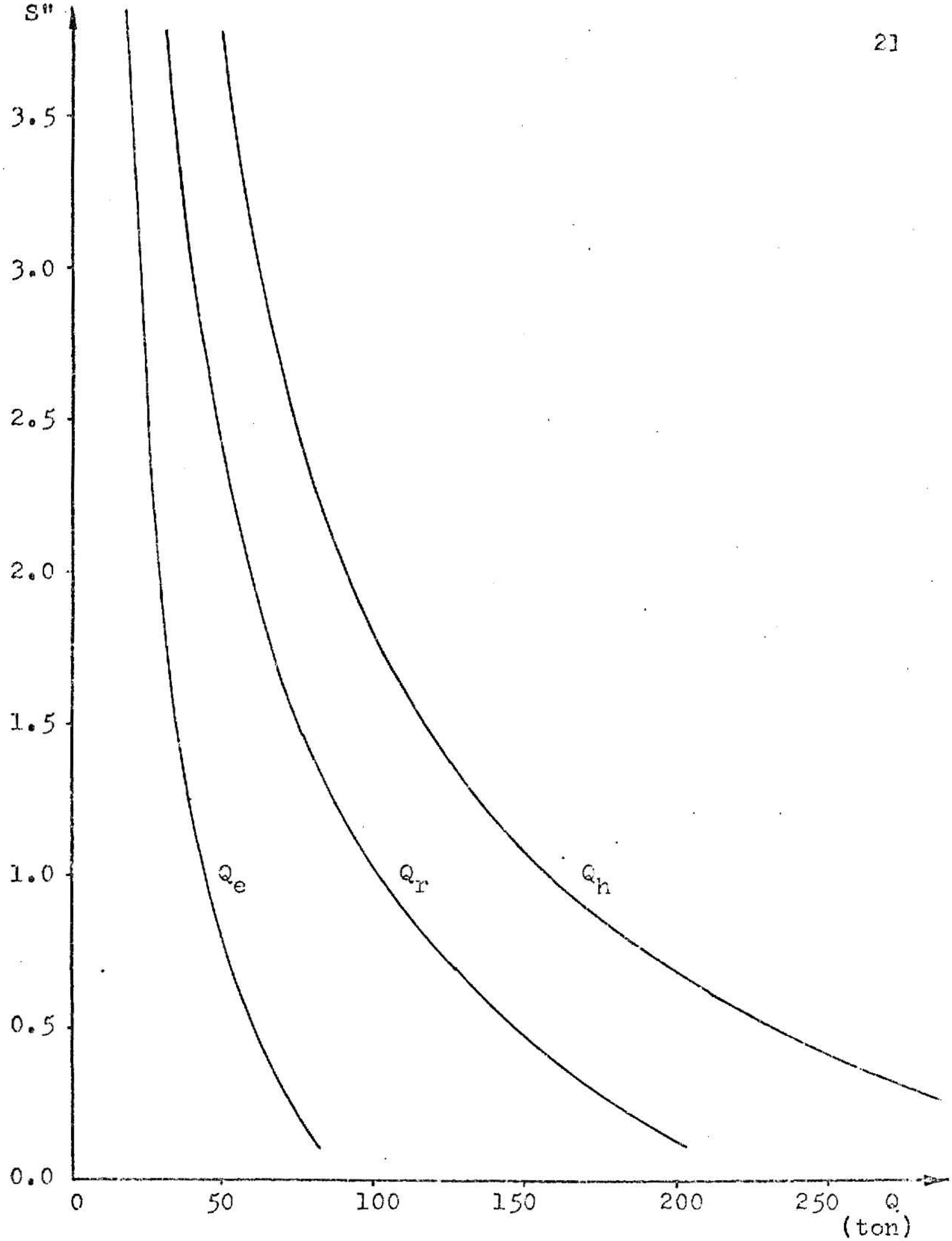

Fig. 7. Curves of bearing capacity (Q) vs. average penetration (S) when L = 100 ft.

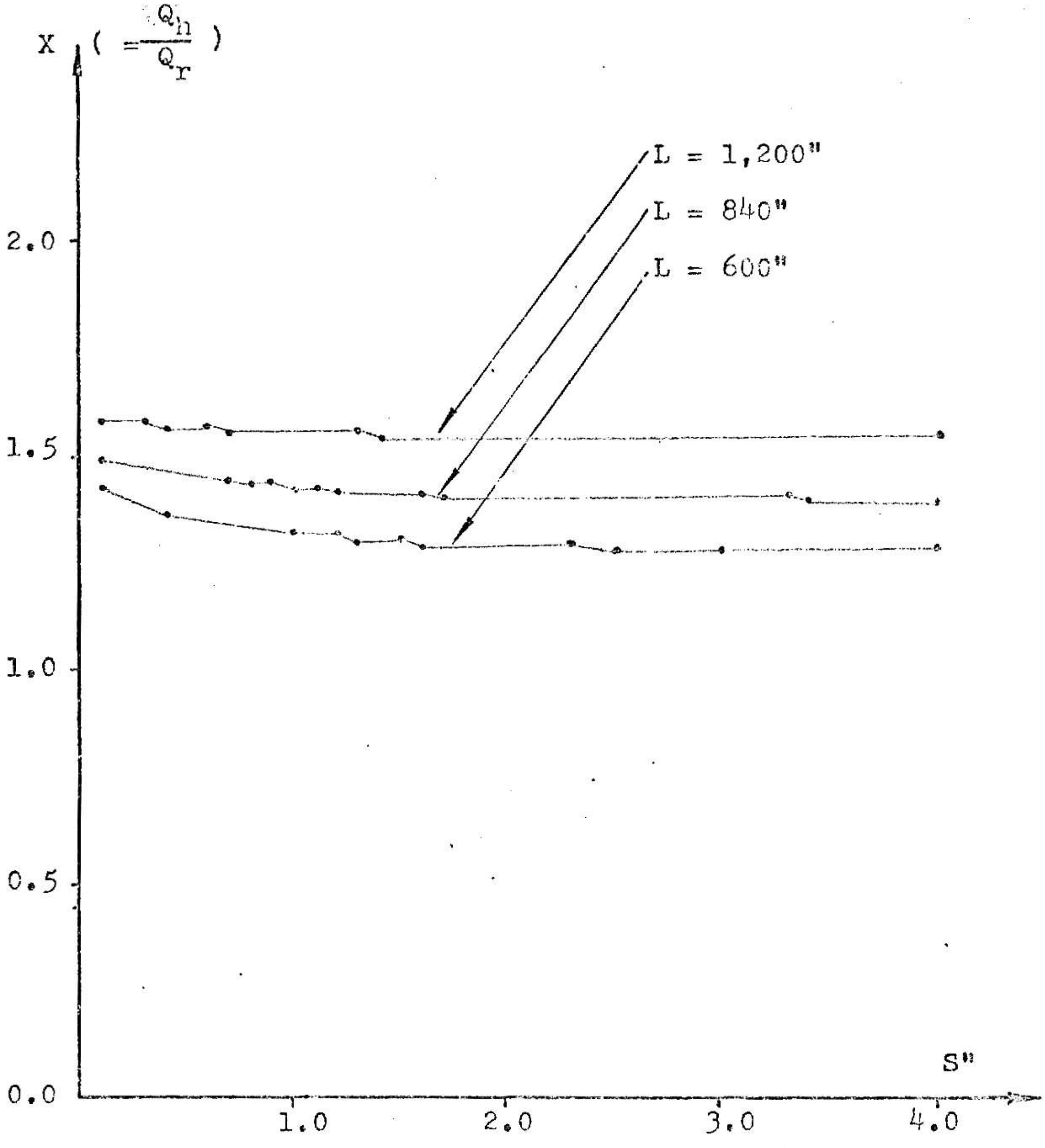

Fig. 8.

The data plotted in Figure 8 show that the value of X is essentially independent of changes in S. It can therefore be assumed that

$$Q_h = \bar{X}\, Q_r \text{ , where}$$

$\bar{X}$ is a constant and lies between 1.30 to 1.60.

By investigating the assumptions that simplified the General formula to Redtenbacker's formula, it seems that the general formula will result in more accurate values of Q for the following reasons:

(a) The real value of C is between 1.00 and 0.67 depending on soil condition of the driving location. It can not be assumed that C = 1.00 since generally, for friction piles, the point resistance is not effective. In addition to this, the area of pile head is small because in practice the area of the cross section of a pile decreases from the top to the bottom. Since we used different values of C for both formulas, it appears that the Q_r value from Redtenbacker's formula is about 30% less than Q_h from Hiley's. From the stand point of safety, Redtenbacker's formula is more conservative.

(b) e is the efficiency of hammer fall. Many experimental values of e_1 for different kinds of hammer were found and are listed in Appendix B of the report. These efficiencies are for drop hammer actuated by rope and friction winch but this figure may decrease when the drop is small and increase somewhat if the drop is very large. For the assumption, although e_1 = 100% is high, using e_1 = 75% does not greatly affect Q and it is clear that e_1 = 100% is a more reasonable assumption than C = 1.

(c) n, Newton's coefficient of elastic restitution, varies from 0.25 to 0.55 according to the materials used in the pile for example, concrete, wood, steel etc. For this test, n = 0.40 - 0.45 for concrete pile without cushion on pile top. Since n^2 = 0.16 - 0.22 and for most cases $W_p > W_h$, the value $n^2 \cdot W_p$ is significant and it seems wise to keep it in the formula rather than letting n = 0.

(d) The value of non-elastic crushing of the pile per blow, (S_f), is a typical experimental figure and is very hard to determine from experimental tables. The only way suggested is to measure this value in the field after each blow of hammer. In concrete pile, for example, we found S_f is almost equal to zero when there is a cushion block upon the pile top and it still be very small when there is no cushioning. Redtenbacker's assumption to let S_f = 0 is reasonable.

The comparison chart on page 22 shows Q_r is about 30% lower than Q_H. It is wise to use Redtenbacker's formula when safety design is wanted.

Actually the general formula may not be more accurate than the simplified one since the general form itself is derived from theory under ideal soil condition which is not true for the practical soil conditions.

Recalling the charts of bearing capacity vs. average penetration per blow, we found Q_e after the Engineering News formula is very low compared to Q_h and Q_r. The tables and charts shown that Q_e is about 25-30% of Q_r for soft soil and is about 30-50% of Q_r for hard soil. If the bearing capacity after Redtenbacker's formula is reliable it is obvious that the factor of 1/6 in Engineering News formula is too conservative.

Going back to equations (14) and (15) on pages 9 and 10, the first type of Engineering News formula is formed from the relation function

$$E_2 = Q_u (S + S_f + S_e)$$

$$= e_1 \cdot e_2 \cdot W_h \cdot H$$

Energy == Energy

By assuming $e_1 = 1$, $e_2 = 1$, and $S_e + S_f = 1$, it gives

$$Q_e = \frac{W_h \cdot H}{S + 1} \qquad \text{for drop hammer.}$$

We can understand that bearing capacity from this formula is high because it neglects energy loss due to impact by assuming $e_2 = 1$ and neglects energy loss due to the hammer dropping by assuming $e_1 = 1$.

A factor of safety 1/6 was used to justify bearing capacity from the original type of Engineering News formula. Once again, that factor is low and it results in an over safe design.

STATIC FORMULA

The evaluation of bearing capacity of pile by static formula (static loading test) is recognized to be the most accurate method for determining safe working loads. Karl Terzaghi understood that any dynamic formula used to estimate bearing capacity of piles, even the most elaborate one, would not be accurate because every dynamic formula involved various arbitrary assumptions with unknown practical implications. Most important of all, the soil conditions in nature vary greatly. Terzaghi concluded that the reliable way to determine the bearing capacity of a pile was to make load tests on "full-size" test piles in the field. The only limitation of the single pile load test was its application to pile groups.

The basic form of static formula is

$$Q_u = Q_f + Q_p$$

Q_u = ultimate bearing capacity of pile,

Q_f = the part of bearing capacity from skin friction,

Q_p = the part from end point resistance.

The value of Q_u can be obtained by the testing results from static load test but the determination of the ratio between Q_f and Q_p is impossible to determine. Q_f is based on the value of skin friction and

$$Q_p = Q_u - Q_f$$