No. H-7

OPENING DISCUSSION

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The use of pile foundations used to be a simple problem and one to which engineers could always resort when in doubt or difficulty. Pile driving was then a pleasant and profitable pastime. Today the problem is becoming so complex and involved that the use of piles may soon have to be abandoned. In the past it was so easy to drive piles to some resistance by some formula, multiply the number of piles by this resistance and thus obtain a foundation. Today we know that our formulae were faulty, that the resistance of one pile is not indicative of the resistance of the entire group, and that the entire group with its surrounding soil mass may be constantly settling. Such is progress.

As I take it, Sections H and I, which I am to discuss, refer to the bearing capacity of individual piles or, in other words, their resistance to penetration in the soil. I shall limit my remarks to this, otherwise the subject becomes entirely too broad in scope.

There are three usual methods of determining the bearing capacity of piles: dynamic driving formulae, static analytical formulae, and load tests.

Driving formulae and static formulae are legion. They have produced, in a way, two schools of thought among engineers. If the theory does not agree with facts the first school would maintain that the the theory is wrong, while the second school would be equally sure that the facts are wrong. There is no doubt that a load test is the only accurate method of determining the actual static resistance of an individual pile, although such load tests probably give no indication of the long time of settlement of the pile. But load tests are expensive, and hence the amount of midnight oil and ingenuity that has been spent on pile driving formulae.

It is evident that a dynamic pile driving formula can be of value only when there exists some relationship to slow penetration. Thus, in permeable, relatively cohesionless soils we may anticipate some definite relationship, while in impermeable cohesive soils the opposite is true.

In a dynamic formula it is essential that the energy losses be determined and then corroborated by field tests. I suggest that there are two types of pile driving formulae which permit this. The first formula is of the type $r = (eWh/s + k)(Y)$, in which k measures the energy loss due to the elastic compression of the pile, the elastic compression of the soil, and other factors.

Our English friends have devised a so-called "inertia gauge" which placed on the top of the pile being driven produces a diagram for each blow of the hammer on a pile, Fig. 1, in which s is the penetration per blow and 2k the rebound of the pile head caused by the elastic compression of the pile, soil, etc.

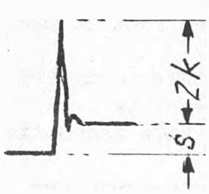


FIG. 1

The other type of formula which permits corroboration in the field is of the Kreuter type, in which h_o represents the maximum height to which a hammer may drop without causing any movement of a pile. h_o may be readily determined in the field, whenever it is possible to vary the drop of the hammer by plotting the heights of fall as ordinates and penetration per blow as abscissae, for several different heights of fall. Drawing a straight line connecting these points gives the term h_o as the intercept on the Y axis (page 220, Vol II). If the line is not straight it indicates that there has been a change of condition during the test and the test should be repeated.

Formulae of the first type are particularly applicable to steam hammers, where it is impossible to vary the height of fall. Formulae of the second type are particularly applicable to drop hammers where the height of the fall may readily be varied. Each of these factors, h_o and k may be expressed in terms of the other:

$$k = \frac{h_o \cdot s}{h - h_o} \qquad \text{and} \qquad h_o = \frac{kh}{k + s}$$

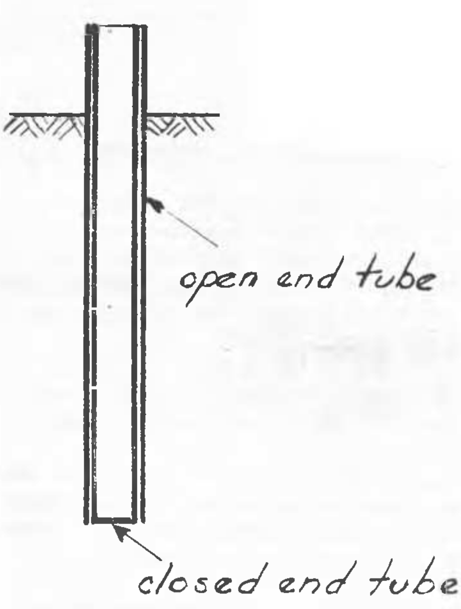


FIG. 2

I am convinced that wide-spread determinations of the values of k and h_o for various types of piles driven into different kinds of soils would give extremely valuable information.

In cohesive soils, where a dynamic formula would not be applicable it appears logical to base the bearing value of a pile on the surface friction along the sides, and to neglect point resistance. Previous to this Conference there has been little data available on the frictional resistance. The data published in these proceedings should be of inestimable value. Let us hope that it is only a beginning.

Whether the bearing value is determined by a dynamic formula or by frictional resistance, the results should be verified, in work of any importance, by static loading tests, with the tests carried to failure. Only by carrying such tests to failure is it possible to obtain complete information.

In connection with loading tests I should like to make one comment. When such tests are made with levers or jacks in such a way that there is uplift on certain piles, it seems to me that there

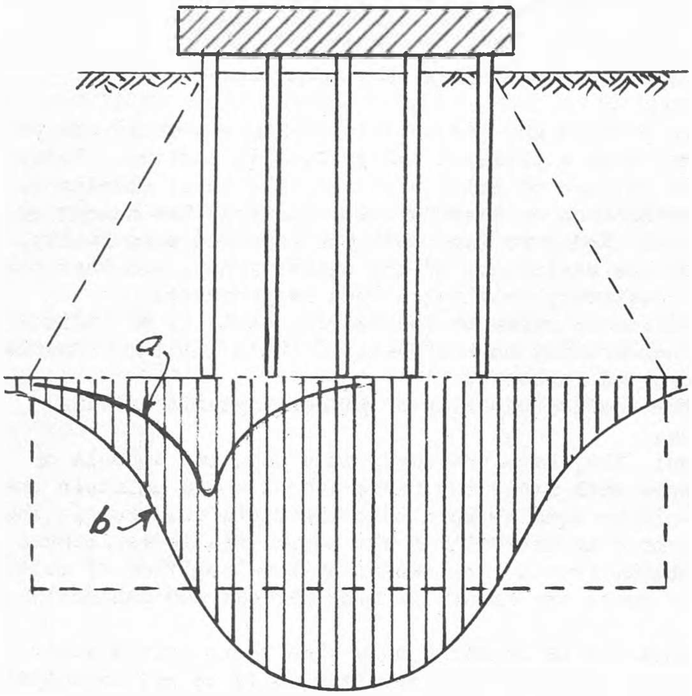


FIG. 3

may be arching of the soil and thus give higher indicated values than would be obtained by direct loading.

There is one method of pile testing and driving which I should like to see carried out at some time and which I think would give considerable valuable data. The apparatus would consist of a steel tube with open end, and in that tube another one with closed end, Fig. 2, so arranged that the outer tube may be driven from the inner tube and the inner tube may be driven entirely separate from the outer tube and loaded in the same way. In that way we should get valuable data as to the frictional resistance along the side, point resistance and settlement.

After determination by driving formulae, frictional resistance, and loading tests, careful consideration must be given to any additional load which may be caused by the consolidation of the upper layers of soil. This has been well described by our friends from the Netherlands. (Paper No. H-3, Vol I).

After this is done, there remains to be determined whether the bearing capacity of the individual piles, when applied to each of the entire group, would cause undue settlement and overload the soil mass under the foundation. This brings up the question of distribution of load on the soil at or below the plane of the points of the piles, a subject about which little is actually known. Using the theory of Boussinesq we can assume that the vertical load on the soil at the plane of the point would be distributed somewhat as shown by curve "a", Fig. 3, for a single pile. For a group of piles these zones of stress would overlap and the resultant distribution of the load on the soil would be somewhat as indicated by curve "b", Fig. 3. The exact distribution depends, as for mat foundations, on the flexibility or rigidity of the structure which rests on the piles and also on the nature of the soil. For a foundation of infinite length the load per square foot on the soil would equal the load per square foot of the structure.

In the new proposed Boston Building Code it is assumed that the load of the structure is distributed through the soil at an angle of 60° with the horizontal and that it is uniform over the area, as shown by the dotted line in Fig. 3. This is not correct, but probably a reasonable safe assumption to use as a basis of calculation.

I should like to express the hope that out of this Conference there may evolve a permanent organization to collect and correlate data on pile foundations, make it available to all interested, and thus encourage and permit rapid progress.

No. H-8 DISCUSSION OF SECTIONS H AND I

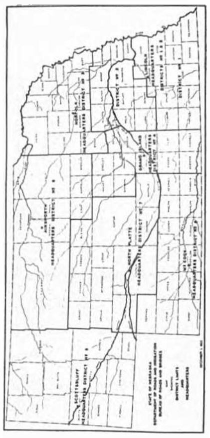
John G. Mason, Bridge Engineer, Nebraska Dept. Roads and Irrigation, Lincoln, Nebraska

Foundations for Nebraska Bridges. Owing to the sand-gravel alluvium sub-soil west of the Missouri River, together with the territory once covered by the glaciers, the Central Middle Western part of the United States presents a peculiar foundation problem to the Engineer. However, many other geographical areas of the United States share this same geological condition. I speak of that new country in the New World which in a geological sense is truly the latest creation of nature; the deposits of the Pleistocene Age.

In the State of Nebraska a recent compilation of data for nearly 10,000 piles driven during a four-year period, shows that only 21 per cent of them reached a rock stratum, the other 79 per cent having been driven into clay, sand, or gravel subsoils. Precisely, there were 33 per cent in clay, 33 per cent in sand, 13 per cent in gravel and 21 per cent to rock.

About 30 to 35 years ago steel piles (fabricated) were introduced in Nebraska. Their use has increased steadily in volume. The tabulation of data for 10,000 random piles, referred to above, shows that 37.5 per cent of all piles used by the State of Nebraska over a four-year period were steel piles. Actually this figure represents about 38 miles of piles, if they were placed end to end (about 203,000 lineal feet).

The prevalent occurrence of this laminated granular subsoil strata to depths varying from 30 to



NUMBER AND KIND OF PILES IN EACH CLASS OF SOIL

Type of Pile	Class of Soil 1	2	3	4
Timber	2687	2304	336	740
8" H	301	706	94	225
10" H	262	202	321	488
12" H	none	none	505	598
Totals	3200	3212	1256	2051

TABLE 1

DISTRIBUTION OF PILE TYPES IN VARIOUS CLASSES OF SOILS

Class of Soil	Pile Types: Timber	8" H	10" H	12" H
1	2657	301	262	none
2	2304	706	202	none
3	336	94	321	505
4	740	225	488	598
Totals	6017	1326	1273	1103

TABLE 2

NUMBER OF PILES BY CLASSES OF SOILS

Class of Soil	Total Number of Piles Recorded	Per Cent
1	3200	33.0 %
2	3212	33.0 %
3	1256	12.9 %
4	2051	21.1 %
Total	9719	100.0 %

TABLE 3

SLIDE Nº 1.

PER CENT DISTRIBUTION OF PILE TYPES IN VARIOUS SOIL CLASSES

Type of Pile	Class of Soil 1	2	3	4
Timber	[illegible]	[illegible]	26.7 %	36.0 %
8" H	9.2 %	22.0 %	7.5 %	11.0 %
10" H	8.3 %	6.3 %	25.5 %	23.8 %
12" H	0.0 %	0.0 %	40.2 %	29.2 %
Totals	100.0 %	100.0 %	100.0 %	100.0 %

TABLE 4

RATIO OF INDIVIDUAL TYPE PILES TO TOTAL NUMBER RECORDED

Type of Pile	Number Recorded	Per Cent
Timber	6017	61.9 %
8" H	1326	13.6 %
10" H	1273	13.1 %
12" H	1103	11.4 %
Total	9719	100.0 %

TABLE 5

RATIO OF INDIVIDUAL TYPE PILE TO TOTAL NUMBER USED 1931-1935

Type of Pile	Number of lineal feet used in Nebraska 1931 to 1934 inc.	Per Cent
Timber	337723'	62.5 %
8" H	80573'	14.9 %
10" H	92289'	17.1 %
12" H	30044'	5.5 %
Total	540629'	100.0 %

TABLE 6

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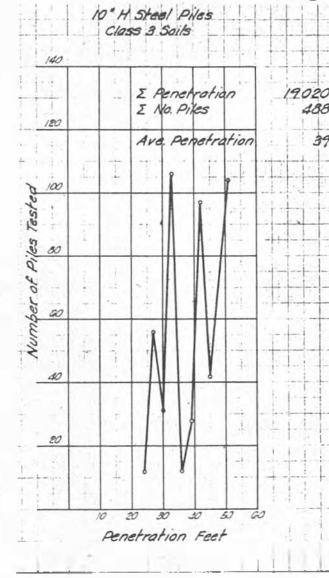

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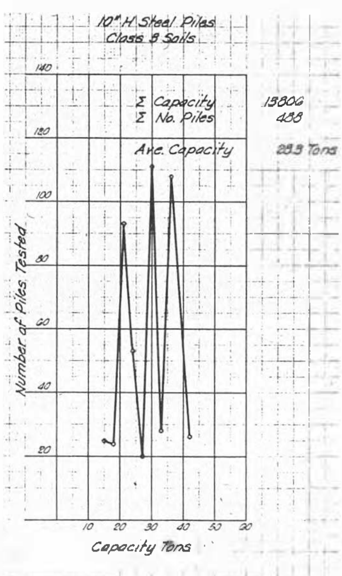

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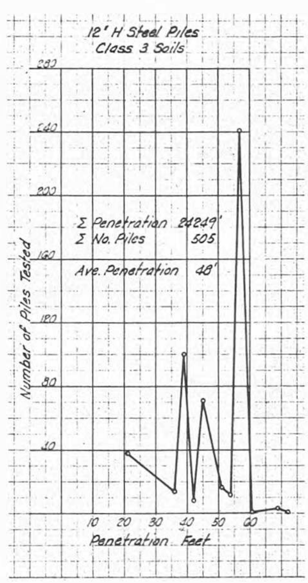

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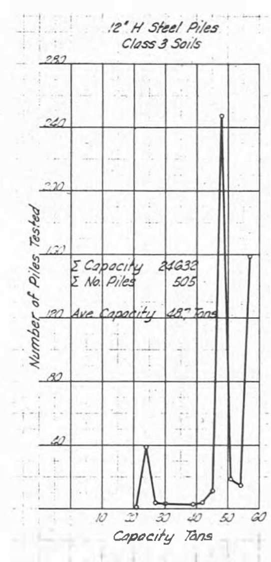

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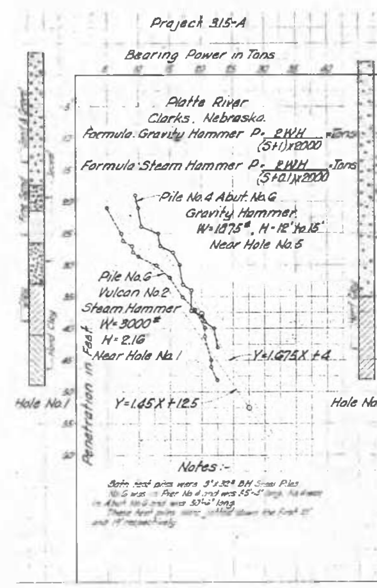

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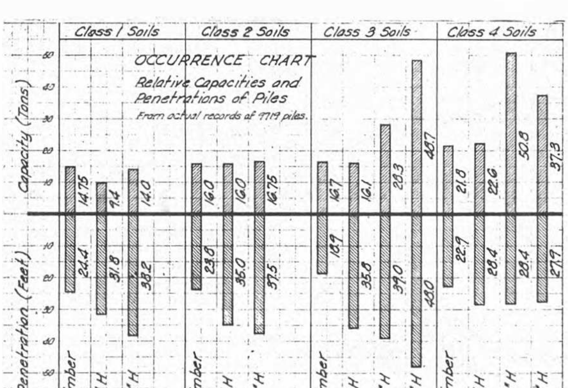

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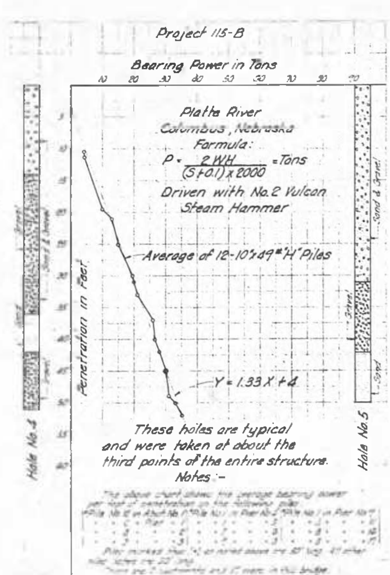

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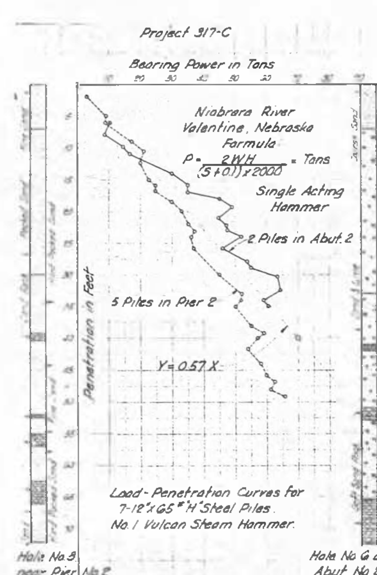

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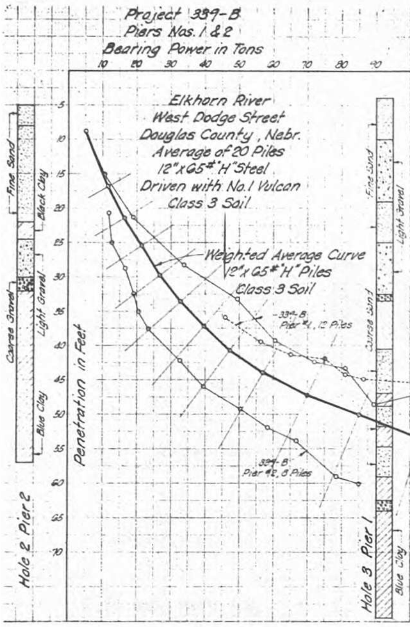

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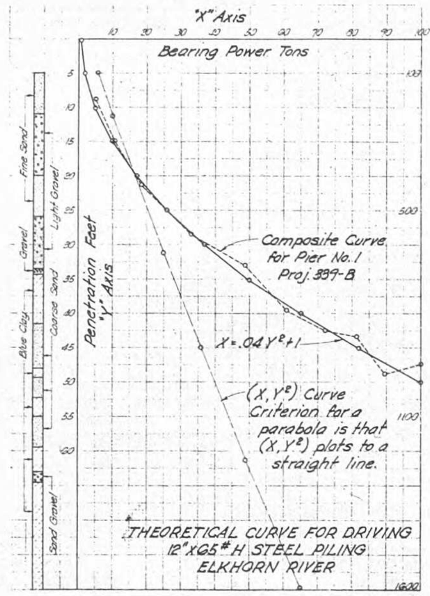

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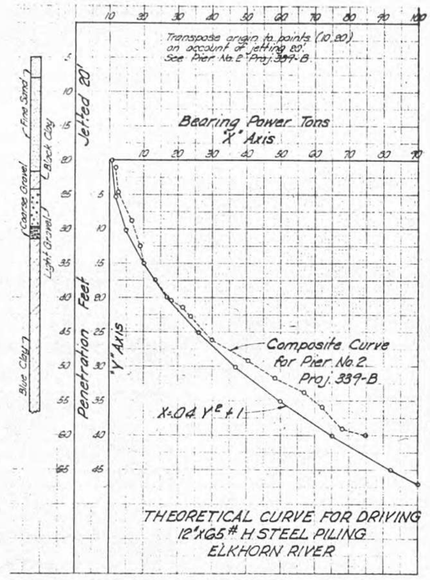

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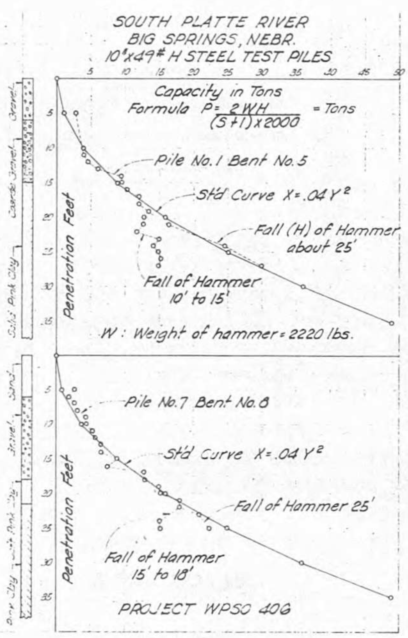

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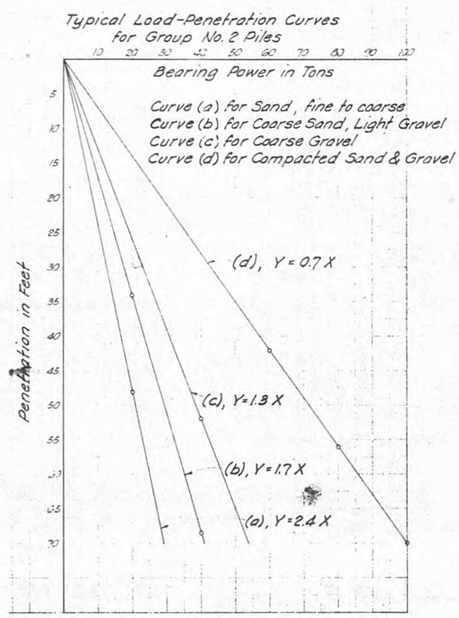

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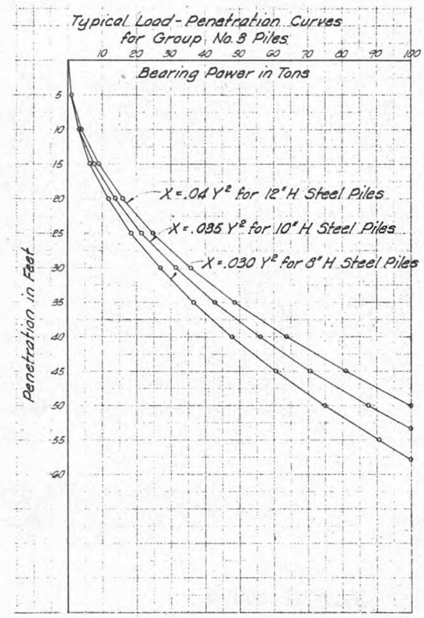

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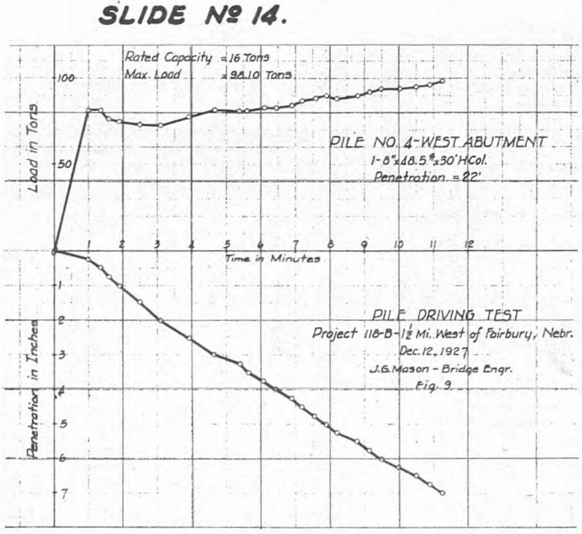

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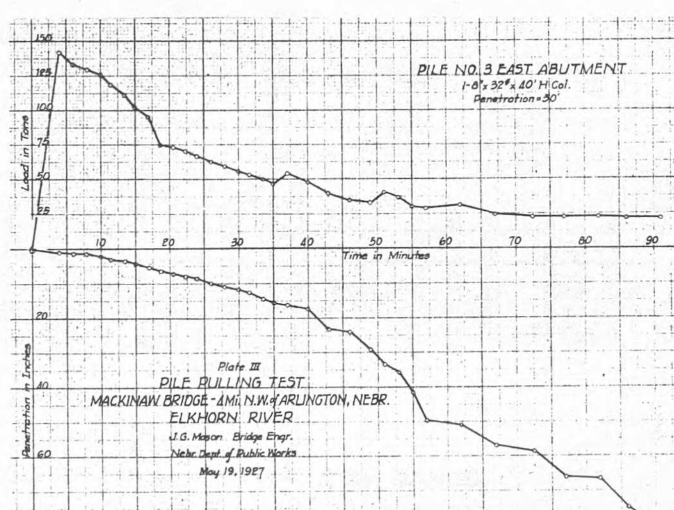
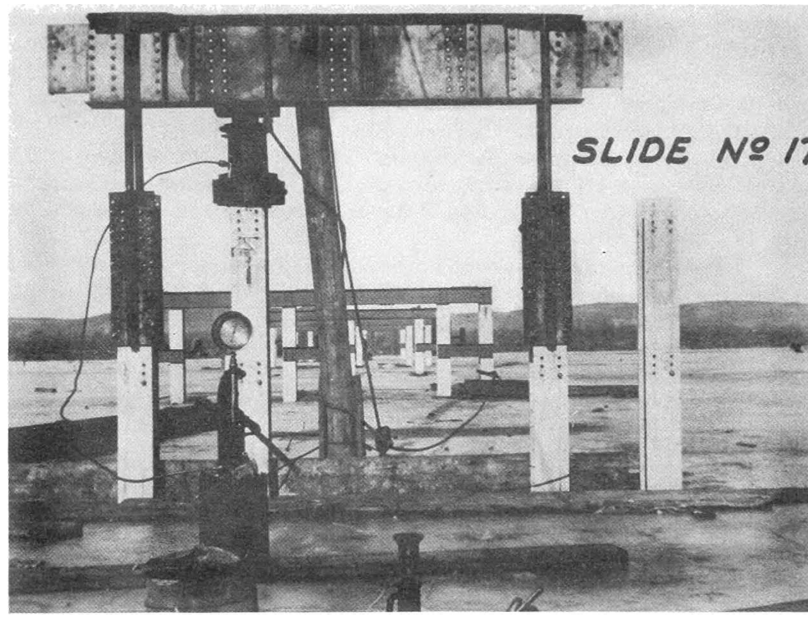


SLIDE Nº 19.

200 feet makes the problem of foundations difficult, for the principal reason that in flood periods the stream bed is susceptible to easy deep scouring action. Therefore, since the bed rock is generally beyond reach, long piles capable of being driven full depth are a necessity. Nebraska's experience over



LOCATION	PROJ. NO.	IDENTITY OF PILE	PILE TYPE	HAMMER W	HAMMER H	HAMMER P	CAPACITY BY FORMULA	MAX. TEST LOAD P: TONS	MAX. TEST LOAD S INCH	TEST LOAD AT ¼" PENETRA	ACTUAL PENETRATION FT.	TOTAL DURATION OF TEST HRS	SUBSOIL CLASS NO.	
West "O" St.	1-2-R	#8 Abut. 2	8" H - 45'	3000	2.46'	1.2"	5.58 Tons	54.0	5/16"	38.0 Tons	33'	24 Hrs	3	
"	1-2-R	#10 Abut. 2	8" H - 45'	"	"	0.7375"	8.67 "	64.0	1 5/16"	42.0 "	33'	24 "	3	Held 47 Ton Load 16 Hrs.
Wood River	3	East Abut.	8" H -				8.90 "	29.0	None	28.86 "	26'		3	Letter to U.S.B.P.R. 4-8-38
Lincoln - Beatrice	18 - Div. III	#5 No. Abut.	10" H - 45'	2040	10'	0.92"	10.60 "	32.7	3/8"	28.90 "		10½ Hrs	2	Anchor Piles Raised
"	"	#6 " "	10" H - 45'	2040			9.72 "	24.0	3/8"			2½ "	2	This pile had previously been moved & Anchor Pile failed to hold
"	"	#4 Anchor	10" H - 45'	"				-16.0	-3/8"				2	Uplifted
"	"	#5 Anchor	10" H - 45'	"				-12.0	-1/8"				2	Uplifted
Cozad	39	#2 Pier 2	8" H - 30'				11.89 Tons	100.0	None		44'	36 Hrs.	3	Penetrated hard. Pile heads battered by hard driving
"	39	#6 Pier 2	8" H - 30'				12.62 "	100.0	None		44'	36 "	3	Clay Bed
Fairbury	118 - B	#6 East Abut.	10" H - 30'	2000	26	0.874"	27.7 "	60.0	3"	69.0 Tons	23'	7 Min.	3	Into Clay Bed
"	118 - B	#5 West Abut.	8" H - 33'	2000	12	0.6"	15.0 "	80.8	3"	69.0 "	26'8"	5 "	3	" " "
"	"	#4 " "	8" H - 30'	2000	12	0.5"	16.0 "	98.1	7"	81.0 "	22'	11 "	3	" " "
"	"	#4 East Abut.	10" H - 30'	2000	26'	0.775"	29.3 "	108.0	1/2"	107.7 "	21'8"	15 "	3	" " "
"	"	#5 East Abut.	8" H - 32'	2000	16	1.0"	16.0 "	-42.3	-1¼"				3	Uplifted
"	"	#3 East Abut.	8" H (32'+24)	2000	12	0.1/8"	21.3 "	80.8	None				3	Anchor Pile Failed
"	"	#6 " "	10" H - 30'	2000	26	0.874"	27.7 "	-40.4	-1"				3	Uplifted
Martin	E146A-1	#3 Pier #8	10" H - 30'	1930	12	0.3"	17.8 "	104.7	1/4"	104.7 Tons		18 Hrs.	3	
"	"	#4 - #9	10" H 30'	1930	13	0.45"	19.6 "	93.1	None			10 "	3	
"	"	#4 - #12	10" H 30'	1930	13.5	0.33"	19.6 "	91.0	None			6 "	3	
"	"	#6 - #6	10" H 30'	1930	13	0.30"	19.3 "	-44.6	-1/2"	-75 Tons -2½"			3	Uplifted
"	"	#3 - #9	10" H 30'	1930	15	0.40"	20.7 "	-87.5	-1/16"				3	Uplift
"	"	#3 - #12	10" H - 30'	1930	14	0.33"	20.3 "	-65.0	-1/16"				3	"
Ceresco	133A	#4 No. Abut.	10" H - 40'	1850			13.64 (2.0) "	17.3	4/16"			1 Hr.	1	Clay
"	"	#7 " "	10" H - 40'	1850			8.15 "	23.1	3/4"	11.36 Tons		2 Hrs.	1	"
"	"	#3A Bent #3	T. Timber - 25'	1850	15	2.92"	7.08 "	29.0	7/16"	20.5 "	18'6"	32 "	1	Timber Pile
Gothenburg	297-C	#12 Abut #1	8" H - 45'	2050	10	0.675"	17.2 "	55.8	7/16"	55.8 "	43'	36 "	2	Sandy Clay Small Creek
Gothenburg	297A	#2 Pier #1	8" H - 55'	2000	12	0.48"	16.5 "	185.0	None		43'	2 Days off and on	3 and into Clay	Clay Platte River
"	"	#4 - #7	8" H - 50'	2000	12	0.50"	16.0 "	173.0	1/8 Set		38'8"	" " " "	3 " "	
Pender	302 C	#7 No. Pier	8" H 50'	2040	7.1	0.981	10.2 "	30.0	1/8"		26'	48 Hrs.	2	Clay and Sand
"	302 C	#6 & #8 No. Pier	8" H 50'	2040			About 10 Tons	-13.0	-1/8"	-17.5 Tons -½"	26'	48 "	2	Uplifted
Cedar Rapids	331 C	#8 East Abut.	8" H 40'	2110	10	4.4"	3.9 "	19.5	5/8"	16.0 Tons	24'	20½ "	1	Yellow Clay
" "	"	#3 East Pier	8" H 30'	2110	10	3"	5.3 "	21.0	5/8"	17.5 "	28'	20¾ "	1	" "
Odell	340A	#2 Pier #1	8" H 30'	2000	10	1.95"	6.8 "	22.5	3/4"	21.0	30'	37 Min.	1	Clay
"	"	#4 Pier #1	8" H 30'	2000	12.5	1.8"	8.9 "	27.5	3/4"	26.0	30'	3 Hrs.	1	Clay
"	"	" " "	-2nd "					27.5	8 3/4"	Process of sinking the			1	
"	"	" " "	-3rd "					33.0	16 5/8"	pile by means of the			4	Into Soft Rock
"	"	" " "	-4th "					31.0	32 3/4"	Hydraulic Jack apparatus			4	" " "
"	"	" " "	-5th "	2000	17	0.89	18.0 Tons	47.0	39 5/8"		33'	5¾ Hrs.	4	
Comstock	344	#3 Pier #1	8" H - 45'	2480	10	2.2	7.8 "	36.5	5/8"	31.5 Tons	37'	23 "	1 ?	Unknown Soil
"	"	#2 Pier #1	8" H - 45'	2480	"	2.3	7.3 "	-18.2	-1/4"		37'	23 "	1 ?	Uplifted
Verdigree	350-5	#3 Pier #2	8" H - 37'	1830	10	1.76	6.6 "	22.5	0"		22'	19 "	1	Anchor Failed
"	"	#6 - #2	8" H - 37'	"	"	1.84	6.4 "	-11.2	-9/16"		22'	19 "	1	Uplifted
"	"	#4 - #4	8" H - 34'	"	"	1.46	7.4 "	17.5	5/8	15.7 Tons	20'	3½ "	1	Clay and Quicksand
"	"	#5 - #4	8" H - 34'	"	"	2.04	6.0 "	-8.7	-1/4"		20'	3½ "	1	Uplifted
Emerson	321-B	#10R - #3	T. Timber 28'	3000	2.42	0.60	10.3	27.0	1 5/8"	9.0 Tons	26'+	38 "	1	Meat Platform Test
"	"	#9L - #3	" 28'	3000	2.42	0.54	11.3	27.0	3/4"	18.0 "	26'+	32 "	1	

(SLIDE Nº 21)

a 40-year period seems to demonstrate that the "H" steel pile is best adapted for such condition.

Nebraska keeps pile driving records of every pile driven and makes auger and wash boring explorations of every bridge site of any consequence. For practical expedience we have divided all subsoils into four great general classes as follows:

Class 1. (Clay soils (except very hard clay and shale)
Class 2. Sand
Class 3. Gravel and packed sand
Class 4. Rock strata and hard shale.

I will now illustrate the development of the above statistics. Slide No. 1: This slide of Tables I to III shows occurrence and distribution of exactly 9,719 piles. Slide No. 2: This slide of Table IV shows the distribution of various kinds of piles in the various soil classes, and Table VI shows 63 per cent timber and 37 per cent steel piles. Slides No. 3 and 4 are typical cases showing the occurrence of 10" H Steel piles in Class 3 soils as to penetration and capacity. Slides No. 5 and 6 show similar data for 12" H piles.

Our investigation covers the occurrence of timber piles, 8" H, 10" H and 12" H steel piles in every one of the four soil classes in which they actually occurred in Nebraska during the four-year period observed. Slide No. 7 is a symposium of all data assembled and studied for that four-year period.

We wish to draw special attention to the relative results of driving the four different kinds of piles in beds of gravel which we designate as Class 3 soils. Thirteen per cent of our foundations are in this class of soil, and 33 per cent are in sand or Class 2 soils. That is to say, 46 per cent of our foundations are in sand-gravel subsoils. This would represent about 60 bridges per year.

Behavior of Piles During Driving. Now I should like to demonstrate the behavior of steel piles as driven in the Nebraska area of the United States. Our experience shows that steel piles in granular soils seem to obey the straight line law as to penetration with reference to loading.

In hard clay and soft shale they seem to obey a parabolic law, the resistance to driving increasing at a faster rate than the penetration. Slides No. 8, 9, and 10 show the load penetration curves for 8", 10", and 12" H steel piles respectively in granular soils (sands and gravels). Slide No. 11 is a composite experimental curve showing the driving of 20 - 12" H steel piles into clay beds underlying the Elkhorn River. Slides No. 12 and 13 show the close relationship of the experimental curve of driving with the theoretical curve of the form $x = b\ y^2 + a$. Slide No. 14 shows similar data for 10" H piles driven in the South Platte River at Big Springs, Nebraska. Slide No. 15 shows a set of straight line empirical curves recommended for steel piles to be driven in granular soils. Slide No. 16 shows a set of parabolic empirical curves recommended for steel piles in hard clay or soft shale subsoils.

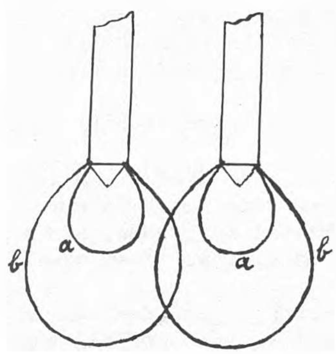
Loading Tests. During the past 8 years we have, for practical reasons, made about 40 hydraulic jack loading tests on various piles in the various soils of Nebraska. Time does not permit of an elaboration of these investigations, except to show the apparatus used and perhaps one or two charts of loading and extraction tests. The following five slides illustrate and summarize the results of these investigations in Nebraska:

Slide No. 17 Martin Bridge Test Apparatus. Slide No. 18, Fairbury Loading Test. Slide No. 19, Mackinaw Pulling Test. Slide No. 20, Photo of the pulled pile at Mackinaw Bridge. Slide No. 21, Tabulation of Loading Tests.

No. H-9 DISCUSSION

R. Pietkowski, Civil Engineer, Delegate of the Polish Assoc. of Struc. Engrs., Warsaw, Poland

We engineers who practice are in such position in our practical life that we must profit from old experiences and old observations. We can use them now critically because in the last few years owing to the studies of Professor Terzaghi and his followers we have learned much in foundation science. But I am of the mind that sometimes we cannot go into small details. I would remember an opinion of one English engineer, expressed some thirty years ago, who said: "I am not quite sure if the formula for retaining walls which I use is at all true, but I have built by that formula about six miles of retaining walls, in different conditions, and they stay. Therefore I can follow that formula and I can recommend it."



We must take care sometimes not to connect unnecessary new troubles with new science. I intend to speak about the "bulb of pressure" in the pile foundations. If we have two next piles we can draw for them the bulbs of pressure which can represent, as Professor Kögler showed us today, 10% of p, or we can draw another bulb of pressure which shall represent, for instance, 1% of p, where p is the unit pressure at the bottom of the pile. In practice we are interested only in the mutual position of bulbs of pressure of such intensity that they can produce deformation of soil. The bulbs of smaller pressure, as for instance the curves b, can overlap each other and from the practical point of view it is no trouble for us. We must take care only that the next bulbs of pressure capable of producing deformations should not be too near to one another. In most cases it is observed that for this aim it is sufficient to give the distance from one pile to the other about 3D - 3.5D where D is the diameter of the pile, and then each pile in a group of piles shall work individually.

When we use the piles, in the up-to-date state of our science, we can calculate their bearing capacity by different formulae. In sandy soils, where we have a rapid distribution of stresses in the ground, it would be suitable to use the dynamic formulae. The most used one of that kind is in my country $P = hQ^2 / e(Q+q)^2$ (formula of Brix); here h is the drop of hammer, Q and q - the weight of hammer and pile, e - the settlement from one drop of hammer. In cases where we have clay ground we must to my mind calculate as when the piles are not driven but bored. We must consider here the resistance of the pile as consisting of two parts expressed by general formula $P = Ap + S(f + c)$, where A and p are bottom area and bottom unit pressure, S is lateral surface of the pile, f and c - lateral friction and cohesion. The sum of $f + c$ can be easily obtained by pulling tests of piles. I suppose that in hard clay soils where the succeeding settlement is almost imperceptible the first dynamic formula is also suitable.

By reviewing the different kinds of piles, I would point out as a result of new soil studies the special advantages of cone-shaped piles in clay soils; the piles of this form by driving go smoothly, similarly as a wedge pressing the water out of the soil; the energy of driving is not so wasted as we observe it sometimes in cylindrical piles; the adhesion of soil to the pile is ascertained and the friction forces are fully developed when in cylindrical piles sometimes a sort of cylindrical hole is formed with feeble adhesion of soil to the pile. In sandy soils the cone-shaped piles obviously easily compact the soil.

Relating to the mentioned in reports Franki-piles I would recommend some caution in driving them near one another; by reason of special method of execution of this kind of piles every succeeding pile can damage by driving by its bottom part the previously executed one.

No. H-10

DISCUSSION

Lazarus White, Consulting Engineer, Spencer, White, and Prentis, New York City

I read some of the papers last night where some of these pile driving formulas were derived, and the result was that my sleep was very much disturbed. I have tried conscientiously to keep up with this subject, and I have read I don't know how many papers in the course of my duties and they all come down to a very simple thing. P, the load that the pile can carry, equals K x S. S is this mysterious penetration on the last few blows. It is all very fine until you come to derive this K. They all have different methods of deriving K. Some of them get integral signs from zero to infinity, over and over again, with a few cosines and tangents of angles, and so forth. They all say, "They have another little K here. Oh well, we'll call that a "C". If you only perform experiments and find this little fellow and apply it to this K, you will get the right answer." And they always do get the right answer. They perform a few experiments and torture this "K" around, and get the right answer.

My patience is about exhausted with that kind of mathematics. There is a great tendency for that to run all through soil mechanics. If we engineers are not careful--and after all this is an engineering subject--the mathematicians will run away with it and then we'll have to fall back on common sense.

Sometime ago I learned from reading Thomas Huxley who was quite a scientist, that one well-performed experiment can explode any amount of mathematics. The mathematics has to agree with nature. Nature has no contract to agree with mathematics.

Now, when you are an engineer and a contractor and have your capital at stake, you don't want to take these formulas at one hundred per cent or else your capital may disappear. That is nowhere more true than in this field of pile driving. If anybody used any one of these formulas and then took financial responsibility for the support of a structure, I don't know what would happen to them. He would have to have a very good lawyer to find a way of ducking responsibility.

In our under-pinning operations we are up against that problem. Here is a building to support, and here are a lot of piles. We drive those anyway. We drive them with a hammer. Then to make sure, we adopt the principle of testing all these piles, individually, hydraulically, with a ram and apparatus just as good as any laboratory, so that we can get the value of single ones and groups. We don't have to depend on formulae. You can take my word for it that the tests reveal that there is no resemblance between the actual result and any one of these formulas, even with correct K, unless that K was tortured into all kinds of forms and was an extreme variable.

I don't think--with all due respect to statistics from Nebraska--that any correlated results based on Engineering News Formula can possible be right.

At first we had the idea that if we tested a single pile, got its value, and then used it, a certain percentage of the value we ascertained would be all right. We found that we were not. We were getting settlements when no computation would show it. Then we went back and tested as groups, and we found that as a group, each individual pile had a far less value than when tested singly. That accounts for the settlements.

Then we went into that problem, and the "Bulb of Pressure". It has been worth a good deal to us because we were able to visualize what happens. We went into that. First of all we gathered all the data and found that this was true. When the bulbs were close to each other they decreased the value of any single one. We could set up a jack on the middle one and put an increased load on, and see the dials of strain gauges decrease on neighboring ones. This is a case where the whole is not equal to the sum of the parts, strange as it may seem. I guess it is not very strange in modern mathematics.

That is absolutely true in pile driving. You have to take everything into account. Every pile driving foundation is a general problem that can't be solved by adding up the parts. You have to know what is below you.

In another case we tested the piles individually and in groups. We thought we were all right, and the building settled. We found that the boring contractor had falsified his results--or at least he reported all clear sand, while below it there was a layer of peat. When the building got to a certain weight the whole building settled. It settled uniformly and it didn't do much damage. It shows that you have to take the underground into consideration in addition to piles individually. Piles in groups and the general problem of soils mechanics cannot be solved by any formula.

No. H-11 DISCUSSION ON THE LATERAL RESISTANCE OF PILES, PAPER NO. H-1
Participants: Paul Raes, Belgium, and A. E. Cummings, U.S.A.

Paul Raes: Very much has been said here about the vertical resistance of piles, but nothing up to now has been said about the lateral resistance. Piles are not always carrying vertical loads.

In 1928 I wanted to give some opinion about piles which had been tipped over by a lateral load. I was looking for a formula and didn't find one. To my knowledge none existed, so I had to find one out, and I worked one out the best I could. To make a formula is very easy, but you must first get some assumptions.

I found that a French engineer--M. Renabenq--assumed, without giving a formula but simply talking about it, that a pile subjected to a lateral load would just rotate around the tip. Other people working on the same subject assumed that the lower part of a pile imbedded in soil and subjected to the same stress would be fixed, supposing that the downward layers were probably so stiff that they would act as a rock. There would be what we call in French, "encastration". This was assumed by Mr. Cummings.

I think that these two assumptions are not correct. I didn't know about the second one in 1928. I made one that seemed to me rather logical and that is that the pile rotates about a certain point situated at a depth y. By using equations of equilibrium you can work out the necessary formulas to find y. Then you can calculate the pile resistance, which I did in a paper appearing in La Technique des Travaux, no. 8 Aout, 1928, page 505. Editor "Societe des Pieux Franki, 196 rue Gretry,Liege.

Unfortunately, I could not test that formula, because in Belgium we knew all about money devaluation long before you did, and the result is that we are very poor. We cannot pay the necessary amount of money to drive piles only for the purpose of testing them and then pulling them out. Americans are still able to make those tests and they have done it very nicely on the Mississippi River. I read a paper by Mr. Feagin appearing in the Proceedings, and telling all about those tests. I did what Mr. White has pointed out with that particular American humor of his that I so much appreciate. Of course, in that formula there appears the famous value of φ, the angle of internal friction. I considered φ as an unknown value. My formula was a sort of equation and I got φ out of that formula and then I saw that at least one test was brilliantly checking the formula and the equations.

But I think that Mr. White has been going a little too far, because if one can do that in one instance, one can not do that when you apply your formula to a lot of piles, which I did, and the different values of φ necessary to get good results were nearly the same.

Following on the paper by Mr. Feagin was a discussion by Mr. Cummings. Mr. Cummings made his assumption and has been doing some very clever mathematics. He made also some very ingenious little tests with a rod imbedded in sand and subjected to a deflection, and he saw that the rod went like this: the upper part went to the left and the under part to the right, and there was a pivot. It gave me great pleasure to see that really I was right, and I thank Mr. Cummings very much for having made that little experiment.

My formula is perhaps very poor, but as it is the only one I think it is the best one.

A. E. Cummings: I want to talk for just a minute about Dr. Raes' paper. He mentioned some work that I have done on the lateral deflections of piles subjected to horizontal forces.

Dr. Raes' two figures are here on the board. This Fig. 1a shows a pivot point at a certain depth in the ground. He referred to an experiment that I performed with some very small, thin steel rods which were imbedded in sawdust. In that experiment I used three rods, one of which was very slim. I think it was about three thirty-seconds of an inch round. Another was about one-eight of an inch square, and the third was a quarter of an inch square. The upper ends of the rods were fixed against rotation.

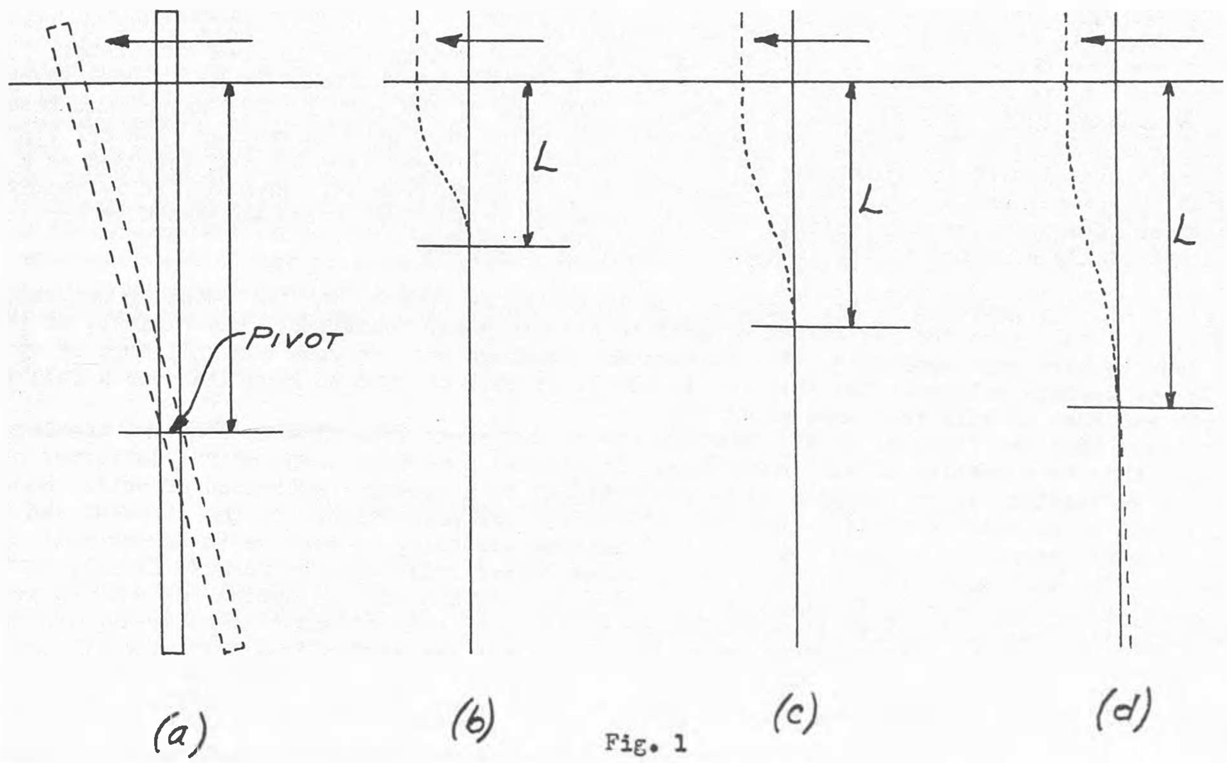


Fig. 1

The very slim rod deflected like that, Fig. 1b, with the lower portion of the rod vertical. The next more rigid rod, Fig. 1c, had a longer deflection curve, but also had the lower portion vertical. The most rigid rod, which was the quarter-inch rod, Fig. 1d, had a longer deflection curve, but did not remain vertical at the bottom. The lower end kicked over slightly.

In the analysis that I published I assumed that some sort of curve like this existed and that the bottom was fixed. But I also included in the analysis the flexural rigidity of the pile itself. The pile is a solid object and it is being moved laterally through another solid. It seems to me impossible that this sort of thing, Fig. 1a, can happen, regardless of the kind of pile that is being talked about. Mr. Raes' analysis would lead to the conclusion that, if this were a one-inch by twelve-inch board, for instance, subjected to a horizontal force, it would behave in the same manner as though it were a twelve-inch by twelve-inch bar of steel. In other words, it would have the same pivot point regardless of the nature of the pile, its shape, its rigidity, or its strength. I think that cannot be right.

I have seen analyses based on the assumption that the pile deflects after the manner of a beam on an elastic foundation. It may be that that is the type of curve that exists when a pile is deflected laterally. My assumption may not be right. However, in calculating the depth to this point, that is, the bent length of the pile, I arrived at an equation of the form

$$L = \sqrt[5]{\frac{216EI\,\omega}{G}}$$

In this equation EI is the flexural rigidity of the pile and G and ω are physical properties of the surrounding soil.

It is my impression--in fact I feel sure--that the manner in which the pile would behave, when it is subjected to lateral forces, depends not only on the soil properties but also on the flexural rigidity of the pile itself, and, regardless of what the deflection curve is, both factors have to be taken into account.

In his conclusions about his theory, Mr. Raes states ".....and no a priori assumptions as to the shape of the elastic curve are necessary." It is my opinion Mr. Raes' entire analysis is based on a very definite a priori assumption about the elastic curve; namely, that there is no elastic curve and that the pile behaves as a rigid body.

Mr. Raes' equations appear to have been derived for the purpose of locating the pivot point on the assumption that such a point exists and on the further assumption that the flexural rigidity of the pile has nothing to do with it. All of the equations contain terms relating to the physical properties of the soil, but there are no terms relating to the flexural rigidity of the pile; i.e., its moment of inertia and its Young's modulus.

Mr. Raes analysis leads to the conclusion that a 1" x 12" board with the 12" face perpendicular to the direction of motion would behave in the same manner as a 12" x 12" timber provided both were embedded in the same soil and subjected to the same horizontal force. It is my opinion that this cannot be true and that the behavior of the pile both above and below the ground depends on the flexural rigidity of the pile as well as on the physical properties of the surrounding soil. This applies regardless of whether the upper end of the pile is fixed against rotation or is free to rotate.

In another of his conclusions, Mr. Raes refers to a photograph published by me. (Proceedings, A.S.C.E., November, 1935, Vol. 61, No. 9, p. 1363.) Only one of the rods in this experiment behaved in the manner mentioned by Mr. Raes. The other two rods which were of much smaller flexural rigidities gave no indication of the existence of a pivot point. The experiment was not intended as a scale model experiment and no quantitative conclusions were drawn from it. However, in my opinion, this experiment demonstrates conclusively that the behavior of vertical piles subjected to horizontal forces is materially affected by the flexural rigidity of the pile.

Paul Raes: I am very grateful to Mr. Cummings for answering my speech, and for pointing out that in my formula, as I only take the resistance of soil to calculate the resistance, the rigidity of piles does not seem to have any importance. But perhaps Mr. Cummings has not read the full copy of my paper. He has perhaps only read the abstract in Vol I. I will be glad to hand him over a full copy of my paper and then he will read more about it.

The reason that the formulas of Mr. Cummings are so different from mine is that Mr. Cummings does not consider this as a problem of soil mechanics. He assumes a certain shape of the deflected pile and then the deflection, y, is supposed to be proportional to a certain resistance of soil. He considers the soil as an elastic solid, where the deflections are proportional to the stresses and the reverse. In other words, he assumes that the soil behaves according to what we in Europe call the Law of Hooke. But soil mechanics assumes that in cohesionless soils--and my formula is only to be used in cohesionless soils--only friction has its part to play and the deflection has nothing to do. Then the deflection has no importance at all, because, assuming that movement is occurring and the angle of friction is reached, the resistance does not increase any more. Then of course, the shape that will be given to the pile after deflection has no importance, and the rigidity, which is only changing that shape has no importance either. That is why, in my formula, the rigidity does not play a part.

But if the soil is not cohesionless and if in a certain region about the pivot where the deflections are very feeble, the limit of actual friction is not reached, of course there Mr. Cummings is quite right, and probably the law of stresses is quite different to what I assumed it was. But still, this can all be included in the factor of safety, and we have no other formula and mine is very simple. But contrary to what Mr. White said, I think that a formula, even a bad one, is better than no formula at all.

No. H-12

DISCUSSION ON SECTION H and I

Dr. W. S. Hanna and Gregory Tschebotareff, Dipl. Ing., Foundation Soils Research Laboratory
Egyptian University, Cairo

Dr. R. Tillman mentioned in his discussion (F-18) yesterday a case of serious settlements of a building in Vienna, founded on conically tapered situ-cast piles. These piles were moulded by means of a massive wooden form driven into the ground and then entirely pulled out. The concrete was then tamped into the hole left free by this form in the ground. A subsequent investigation of this case had shown that the concrete of the piles had a smaller section at the level of a deeper lying clay layer, presumably due to the lateral pressures developed during the driving of the adjoining piles and the resulting lateral displacements of the clay.

This discussion, which followed our lecture and made a general reference to it may have created an erroneous impression that the situ-cast piles of the buildings in Egypt we referred to were of a similar type. We therefore have to make the following remarks:

1. None of the buildings we referred to in our lecture had piles of the type mentioned by Dr. Tillman. This type has not been used in Egypt during the past 8 years. The various situ-cast piles in current use in Egypt of over 6 meters length are formed in the ground by means of hollow cylindrical steel tubes gradually pulled up as the concrete is rammed in. With this method there is always some concrete inside of the tube, which is kept dry, and a hole in the ground is not left free.

2. There is no evidence so far available of any kind that a partial pinching off or any other damage to the not quite hardened and not encased, by permanent steel tubing, concrete of a pile may be

caused in our soils by lateral displacement of clay due to the ramming in of adjoining piles.

In buildings in Egypt there are seldom more than 3 or 4 piles in one group and their spacing is, as a rule, not under 1 m 20 (i.e. 4 ft) from center to center and is often greater. Load tests are, as a rule, carried on piles chosen from such fully completed groups and their results have not so far revealed that any of these piles were damaged in any manner. Also no excavated piles from such groups, (instances of which have been recorded sometimes), have so far shown signs of any pinching off or of any other damage.

3. It can be seen from the drawings of our paper in Vol I and of our lecture, that the settlement distribution in plan of all pile foundations on Egyptian clays so far observed followed the intensity of stress distribution and did not show irregularities, which could be attributed to damaged piles. Irregular distribution of settlements occurred only with piles resting on sand,--where pinching off of the unhardened concrete by the driving in of adjoining piles is to be feared much less,--and which distribution appears to be due to the irregular nature of the river sand deposits.

All this question has always been in our minds and the fact that such pinching off of unhardened and unencased concrete piles seems to have been observed on clay in some other localities outside of Egypt, with the resulting use of more expensive situ-cast piles left encased by steel in the ground, proves the necessity of a further careful investigation of the matter and of a comparison of the physical properties of clays in the localities where such different effects of the driving of adjoining piles on non-encased situ-cast piles have been observed. Such comparisons might lead to a disclosure of the causes of the assumed different behaviour of various clays in this respect, or of other factors of importance, and subsequently allow the choice of rationally economic designs of foundations in new localities where situ-cast piles had not been extensively used so far.

No. H-13

DISCUSSION

Carlton S. Proctor, Consulting Engineer, New York City

Only as a corollary to Mr. Crandall's very able and excellent paper this morning on pile-driving formulas, (H-7) I want to offer the thought that the development of a new, practical and workable formula should include a determination of the stresses to which the piles are subjected during any stage of driving, and that there are cases continuously where it is desirable to penetrate to an underlying material through a hard crust where it is entirely possible, or probable, that the pile is overstrained during driving.

Pile driving formulas in the past have been considered essential for the purpose of field determination of pile bearing capacities. A point which has been entirely overlooked is the necessity of knowing the driving resistance, irrespective of materials penetrated. In order to properly design a pile and to assure that the pile does not suffer from over-stress during driving, it must be recognized that the most severe treatment or load applied to a pile may take place during the driving operation.

At the present time there are numerous pile formulas which are used with presumed factor of safety. It is not unknown that piles have been driven and presumed capable of carrying loads with such a factor of safety, while at the same time the product of the bearing capacity as so assumed and the assumed factor of safety would result in actual driving resistance considerably in excess of the ultimate strength of the pile being driven. Such an assumption must certainly be contrary to the facts and must disprove the correctness of the formula used.

Several well-known authorities, such as Dr. Terzaghi, E. P. Goodrich, and J. Stuart Crandall, have written excellent papers on this subject, and appear to be in general agreement as to a general type of formula. I do not believe as yet that the subject of the strain of the pile while being driven has been given adequate consideration.

No. H-14

DISCUSSION

Kjetil Haugeto, Designing Engineer, M. W. Kellogg Co., New York City

During my experience I have designed a number of piling foundations located in regions where earthquakes occur--in California, Japan and Manchuria.

Due to lack of confidence in the general practice in design of these foundations, I have treated them with a certain caution. When an earthquake occurs the oscillating motion of the subsoil may reduce the friction, especially at the upper portion of the piles, as shown in Fig. 1.

In the design I have therefore preferably used mat foundation that would partly bear on soil, and rigid struts between the different pile groups. Some of these foundations have been in use for six years, and have resisted severe earthquakes without any unfavorable reports.

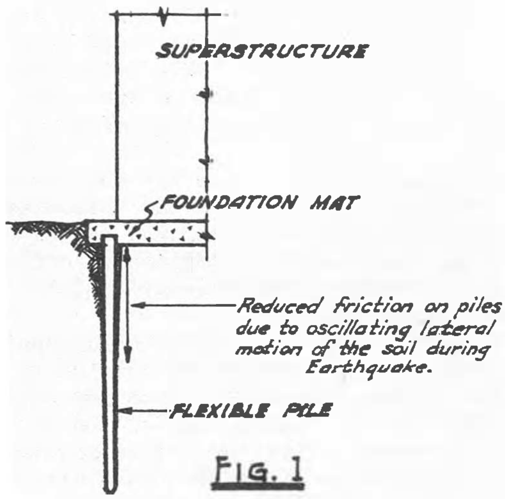


FIG. 1

Due to the increasing heights of buildings and heavier equipment in the industry this may be an important matter for safety. It would be interesting if some of the engineers present from the earthquake regions would give their opinion on how friction is affected by earthquakes.

No. H-15 DISCUSSION (By Letter)

SOME REFERENCES TO PILE DRIVING AND THE SUPPORTING POWER OF PILES IN BRITISH TECHNICAL PUBLICATIONS

Robert F. Leggett, Canadian Sheet Piling Co., Ltd., Montreal, Canada

Discussion at the Conference sessions, and informal conversations during the week of the meeting, indicated that British technical papers dealing with pile driving and the supporting power of piles were not well known. As the Proceedings already include some short bibliographies, it is thought that a list of the most important of these publications may be of interest and of use. The following titles and notes are therefore submitted:

1. "A formula and diagram for ascertaining the resisting power of piles"; by A. C. Hurtzig; T.L.E.S. Vol. 7, page 111, 1886.

2. "A new method for determining the supporting power of piles"; by Franz Kreuter; P. Inst. C.E. Vol. 124, page 373, 1895.

3. "Pile driving and the supporting power of piles"; by Henry Adams; T. Conc. Inst. Vol. 8, 1916.
(The above mentioned papers are concerned largely with the evolution of formulas).

4. "The impact of imperfectly elastic bodies and the effect of the Hammer Blow in Pile Driving"; by A. Hiley; T. Soc. Engs. 1923.

5. "A rational pile driving formula"; by A. Hiley; Engineering, issue of 29th. May, 1925.

6. "Pile Driving Calculations with Notes on Driving Forces and Ground Resistance"; by A. Hiley; Struct. Eng. Vol. 8, July and August, 1930.
(The above mentioned three papers all deal with the "Hiley" formula, mentioned so often at the Conference).

7. "Pile Formulas"; by J. H. Nicholson; S.E.P. Inst. C.E. No. 62, 1928.
(A comparison of the Wellington, Dutch and Sanders formulas, based on test results, given in useful table.)

8. "The Design of Piles"; by G.B.R. Pimm; S.E.P. Inst. C.E. No. 78, 1929.

9. "Pile driving and the Supporting Capacity of Piles": by R. Bennett; S.E.P. Inst. C.E. No. 111, 1931.
(Evolution of another formula)

10. "The Consolidation of Piles"; by A. C. Vivian; C. Eng., page 23, November issue, 1931.
(An experimental study of the "setting up" of piles during driving, after they have been left untouched for short intervals of time)

11. "Piles and Pile Driving"; a Book of 214 pages, by A. C. Dean, published by Crosby, Lockwood & Son Ltd., London, 1935.
(Generally descriptive, with a good chapter on "The Theoretical Carrying Capacity of Piles" in which 45 different pile formulas are discussed and analysed.)

12. "The Resistance of Piles to Penetration"; a Book of 138 pages by R. V. Allin, published by E. and F. N. Spon Ltd., London, 1935.
(Essentially a book of tables based on the use of the Hiley formula giving the Resistance to Penetration of piles which, as the author points out, is not the same thing as the Supporting Value which can be used in design.)

13. "The Actual Bearing Value of Piles"; C. Eng. page 210, July issue, 1935.
(A most valuable note, giving 48 references to technical publications which include actual test pile data, the main essentials of which are presented in a comprehensive table.)

14. "Pile Driving Formulae as Applied to the Reinforced Concrete Pile"; by R. V. Allin; Conc. and Const. Eng., page 357, issue of July, 1935.

15. "The Behaviour of Reinforced Concrete Piles during Driving"; by W. H. Granville, P. Grime, and W. W. Davies; J. Inst. C. E. Vol. 1, page 150, December, 1935.

(A technical paper, presented to the Institution of Civil Engineers, and discussed, describing researches carried out at the Building Research Station, Watford, of the British Government, in collaboration with the Federation of Civil Engineering Contractors, in order to devise methods of finding what a pile will stand when being driven without being damaged. Describes the development and use of a piezo-electric strain recorder, using cut quartz crystals.)

16. Note and Practical Suggestions on Pile Driving; J. Inst. C. E. Vol. 1, page 587, April, 1936. (A progress report on the newly reconstituted joint Sub Committee of the Institution of Civil Engineers on Pile Driving, indicating present problems and directions for future research work.)

The above list is not exhaustive, but it does indicate most of the important British publications on this subject. The references include papers of varying value, some being included mainly to indicate regular periodicals which may be of general interest, abbreviations being used to denote these as follows:

P. Inst. C.E.: Minutes of the Proceedings of the Institution of Civil Engineers, covering about one hundred years of British engineering and terminating with Volume 240.

S.E.P.Inst. C.E.: Selected Engineering Paper of the Institution of Civil Engineers, published as separate pamphlets.

J. Inst. C.E.: Journal of the Institution of Civil Engineers, the new form which all the Institutions publications now take, started in 1935, following on from Volume 240 of the Proceedings to be issued eight times a year, and generally available through technical booksellers.

T.L.E.S.: Transactions of the Liverpool Engineering Society (one of the several provincial societies in Great Britain) issued annually.

T. Conc. Inst.: Transactions of the Concrete Institute (London) the precursor of:

Struct. Eng.: "The Structural Engineer", a monthly journal, and official organ of the Institution of Structural Engineers (London).

T. Soc. Engs.: Transactions of the Society of Engineers (London).

Engineering: "Engineering" a weekly technical journal (London). "The Engineer" (London) a similar publication often presenting relevant information.

Civ. Eng.: "Civil Engineering" (London) a monthly journal to be distinguished from the journal of the American Society of Civil Engineers.

Conc. & Const. Eng.: "Concrete and Constructional Engineering" (London) a monthly journal described by its name.

No. H-16 DISCUSSION OF PAPER H-5 (By Letter)

Dimitri P. Krynine, Research Associate in Soil Mechanics, Yale University, New Haven, Connecticut

The object of this discussion is to complement the information furnished by Mr. Lozovsky so far as "pile driving" or "ramming" curves are concerned. The term used by the writer is "energy - penetration curves" since he constructs them by plotting the accumulated mechanical energy, E, spent in pile driving, against the total penetration, L, of the pile at a given time.

During the construction of a bridge in Stratford, Conn., in 1933 piles were driven at the bottom of an excavation (elevation of the bottom + 62, see Fig. 1); the water table was at + 61; and the material consisted of fine saturated sand, fairly homogeneous. The piles were of creosoted yellow pine, and their average diameter was 10". An experimental pile (Exp. P. in Fig. 1) driven before the construction with a rather light McKiernan Terry double acting hammer 9-B-2 did not reveal a regular "energy-penetration" curve; while in the case of other piles observed during the construction (hammer 10-B-2 made by the same firm) it was found that "energy-penetration" curves were of parabolic shape. Foot-pounds of energy per blow are 8,200 for the former and 15,000 for the latter type of hammer. Energy of fall alone would be 2,000 lb-ft and 4,167 lb-ft, respectively.

The equation of a parabolic "energy-penetration" curve is:

$$E = \frac{1}{2} A.L^2 \tag{1}$$

where A is a constant. After differentiation, equation (1) furnishes:

$$dE = A.L.dL \tag{2}$$

This means that the amount of energy which is spent to drive the pile for a distance, dL, is proportional to the product A.L, or to the driven length of the pile, L, since A is a constant. In other words, if the "energy-penetration" curve is parabolic, the pile resistance is proportional to the driven length, L, and may represent either skin resistance or deformations of the pile proportional to its length or both. The point resistance in such a case is very small, if at all. Equation (2) may be rewritten under the form (3):

$$\frac{dL}{dE} = \frac{1}{A.L} \tag{3}$$

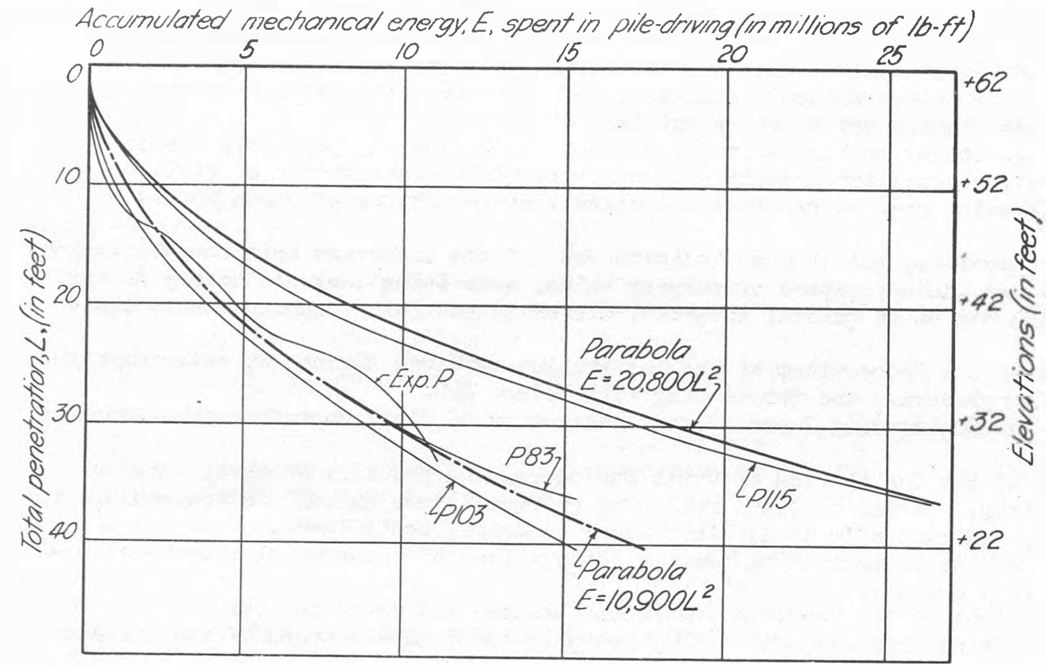


Fig. 1 Energy-Penetration Curves showed Parabolic Shape in the Case of Stratford Bridge.

If the amount of energy spent per blow is constant, as is the case of steam hammers, equation (3) would mean that, under the given assumption, the penetration per blow is inversely proportional to the length of the driven portion of the pile, L. If the pile is being driven into a homogeneous deposit, and after a certain number of blows the parabolic character of the "energy-penetration" curve has been revealed, it may be predicted, obviously within certain reasonable limits, after how many more blows the penetration per blow would reach a given value (for inst., one-tenth of an inch per blow).

An "energy-penetration" curve in the form of a straight line would have for equation:

$$E = A_1 \cdot L \tag{4}$$

or, after differentiation:

$$dE = A_1 \cdot dL \tag{5}$$

Equation (5) would mean that the dynamical resistance in such a case is chiefly due to the presence of a more or less constant point resistance, the skin friction being relatively negligible.

The observations by the writer referred to, were published in the Annual Report of the Connecticut Society of Civil Engineers for 1934. (pages 70--81). According to a quotation by M. Singer ("Der Baugrund", 1932, page 159) parabolic "energy-penetration" curves were obtained in an observation made in Germany about 1928. The curves flattened sensibly at a depth corresponding to a change of geological conditions: a flatter parabola corresponds to a harder layer. Another example of such parabolic curves is given by H. Press ("Der Bauingenieur," vol. 14, September 15, 1933). E. Burkhardt in Germany ("Die Bautechnik", vol. 9, April 17, 1931) drove an experimental pile to a depth of about 20 feet and recorded the accumulated number of blows. When the pile passed through the upper loamy and gravelly layers, the curve in question was rather parabolic; but it became practically straight in the lower layers of coarse gravel and soft sandstone. This may mean that in similar soils the point resistance prevails.

To summarize these considerations, it seems possible to judge from the shape of an "energy-penetration" curve, whether the skin friction or the point resistance prevails in the given case of pile driving.

H-17

DISCUSSION OF PAPER NO. H-2 (By Letter)
CALCULATION OF PILES ACCORDING TO THE THEORY OF THE LONGITUDINAL BLOW

B. B. Kretschmer, U.S.S.R.

1. The references to the preceding investigations are absent.

2. The reference on the classical theory of the calculation of the piles is not clear. It appears that the discourse is about the new theory of Professor Doctor N. Gercevanov, because the preceding investigators have not considered the circulation of waves in the calculation of piles.

3. Dr. Gercevanov, for the case with the lower end hardly strengthened, has applied the Saint-Venant's resolution, and made a discreet transition to the conditions of the driving down of piles. As Saint-Venant, N. Gercevanov has thought, that the shock load is hard, because it was necessary to study the special conditions of the contact and the part of the energy which is usual for the elastic oscillations.

4. A. A. Kanshin and A. S. Ploutalow have performed for the shock of free rods the modelling on the particular case the St. Venant's resolution, with the same methods, that were in Annalen der

Physik, 1909, by Ramsauer published, but they have not made the reference to him.

5. The application of the Ramsauer's models in the driving down of piles is technically unsuitable by reason of the high overstrains in the head of the pile and of the large speed of waves.

No. H-18

NOTES OF PROCEEDINGS
Committee on Bearing Value of Pile Foundations, Waterways Division,
American Society of Civil Engineers, June 24, 1936, Faculty Club, Harvard University
Published by Authority of American Society of Civil Engineers

Present at Meeting

Members of Committee

R. E. Bakenhus, Chairman.
J. Stuart Crandall
F. T. Darrow
Glennon Gilboy
J. G. Mason
C. S. Proctor
Karl von Terzaghi
Lazarus White

Secretary, Executive Committee, Waterways Division:
Ralph H. Mann

Members of Committee on Concrete Piles, Construction Division:
J. Wright Taussig, Chairman
A. E. Cummings

Member of Committee of A.R.E.A. on Piles:
Wendell R. Wilson

Foreign Engineers, Guests of Committee:
Aage E. Bretting, Denmark
Giovanni Rodio, Italy
Robert F. Legget, Canada
Dr. Paul-E. Raes, Belgium

Guest of Committee:
F. E. Schmitt, Editor of Engineering News-Record.

Guests of Chairman:
O. R. Egge
D. M. Burmister

1. Stenographic record was made of the discussions at the Committee meeting. No votes were taken. The following report, based on the stenographic notes is believed to represent the consensus of opinion of those present at the meeting except where differences of opinion are brought out. In a general way the discussion was based on subjects into which the Committee work was tentatively divided.

2. Glossary of Terms, Nomenclatures and Definitions is a subject mentioned in the opening address to the International Conference on Soil Mechanics and Foundation Engineering by its President, Dr. Terzaghi, as being important. All at the Committee meeting seemed to agree to this. It was pointed out that the same words have different meanings in different parts of the United States. Such lists of terms and definitions should be sent in at an early date and be submitted to Committee membership for comment. (Refer to definitions adopted at Shanghai and in Göteborg, Sweden.).

3. It was suggested that in some cases illustrations or diagrams would be desirable in making up definitions, as is done in some of the larger dictionaries.

4. Safe Bearing Values is a term difficult to define in mathematical values. It depends on the use. In one case the owner had machinery and did not want more than one-hundreth of an inch settlement. Cases of greater settlement were mentioned.

5. Jurisdiction of the Committee on Bearing Value of Pile Foundations is derived from its appointment by the Chairman of the Waterways Division, American Society of Civil Engineers, at the request of the Society central administration. Overlapping with the work of the Society Committee on Earth and Foundations was mentioned. The special purpose of the Committee on Bearing Value is to prepare a Society Manual or Code while the Earth and Foundations Committee is a continuing Committee. It was the consensus of opinion that the Committee on Bearing Value should continue including horizontal or inclined loads on piles as well as vertical and that the two committees should co-operate and exchange information, each in carrying out the purpose for which appointed.

6. Determination of Settlement. The importance of reliable bench marks was brought out. In the case of the Albany Telephone Building the records stopped after about one year as the bench marks disappeared. Other cases have been mentioned. Cases of general settlement of large areas as at New Orleans and Boston and other cities were mentioned, where in one case records of settlement were difficult if not impossible to get. Reliable bench marks that do not themselves change elevation are evidently necessary, and evidently more than one should be used. A type of bench mark devised by Dr. Terzaghi

was mentioned by him. Settlement may be evidenced by cracks in buildings, change in slope of floors or other horizontal lines, or vertical lines, which may or may not be due to settlement, equal or unequal, of foundations.

7. Pile driving records. The following were mentioned as important: soil profile to sufficient depth below pile point, material length and cross-section of the pile, depth to which driven, elevation of point and of cut-off or top, penetration under blows of hammer, actual loading test, location plan and identifying numbers, elastic behavior of pile under hammer blows, use of pile cap cushion or follower, weight and type of hammer, weight of the pile. Note time of driving and test blows and whether an interval of time has elapsed. Mr. Legget, guest, mentioned a form of pile driving record devised by the Institution of Structural Engineers of Great Britain.

8. The desirability was mentioned of some simple recording instrument that would make a time-penetration record of the action of the pile under the hammer blow and would show by curve the penetration and rebound curves. Methods used by Captain McKay and Commander Angas at San Diego and by a Vienna firm of contractors were mentioned.

9. Mr. Legget, guest from Canada, mentioned a successful series of tests of behavior of piles while driving, made by builders of the substation of the British Government at Garston, England, using a crystal in connection with an electric current. See discussion in the current volume of the Journal of the Institution of Civil Engineers containing also a collection of pile driving data.

9. Water Jet in Pile Driving. Used considerably in sandy soils. Valuable in getting a pile down to predetermined depth without injury to the pile from excessive driving. Driving of sheet piles on the sea coast was mentioned. On the Canadian Rivers piles are jetted in quick-sand bottoms and could not be got down any other way. The piles went down straight with one jet pipe outside the pile. The ordinary practice in this country and in Europe is to drive piles to a satisfactory penetration after they have been jetted. The effect of jetting further piles in the vicinity of those already tested must be noted, particularly if the later piles are jetted deeper. It was also brought out that a pile jetted to say 40 feet and driven to 42 feet would carry more than a pile jetted to 20 feet and driven to 21 or 22 feet. In using a water jet to sink a pile, judgment must be used because the pile will go down as long as the sand is in the state of a liquid mass and may go down further than necessary to develop the desired load. It may be tested after the sand has settled in place and then if necessary, driven and tested again. Good judgment must be used. A case was mentioned on the Platte River where piles could not be driven to the pre-determined depth as it could not penetrate a certain hard clay stratum. Then with a jet a hole was punched through the clay stratum permitting fine sand from below to escape into the river. Subsequently, the piles by use of the jet were carried down as there was an escape for the confined sand which made the jet effective when it had not been before. Another case brought up piles in a highway to Coney Island. They had been jetted, driven and tested satisfactorily. Some one later doubted their stability and under a second driving they went a foot to three feet further under the same resistance. It was always thought the jetting of adjacent piles had injured the piles driven earlier.

10. Pile Driving Hammers. Energy going into the pile determined from the number of blows per minute. Firms making hammers have empirical formulae for energy per blow. In case of a double acting steam hammer an indicator diagram on the steam cylinder for the downward blow would be necessary.

11. It was mentioned that the drop hammer could be used to gain information as to the elastic behavior of the pile by striking blows of increasing height until the fall is determined where the rebound becomes less than the total penetration, that is, the point where a permanent penetration first begins.

12. Friction loss in the leads of the drop hammer and in the drum overhaul and all losses in friction between the falling hammer and the motor must be taken account of in determining energy of the hammer blow, particularly in using a formula to determine safe load.

13. There is not much loss in the single acting steam hammer such as the Vulcan. There is friction of steel against steel or cast iron, well lubricated. The losses in a double acting steam hammer are more difficult to determine, in part due to differences in steam pressure at the hammer and at the boiler.

14. One member remarked that the drop hammer was almost obsolete, with less than five per cent of all piles driven that way, and that either the single acting or double acting steam hammer was almost universally used. Another member brought out that there is a large volume of bridge work done in the rural districts and that the drop hammer due to its portability and simplicity, will continue in use for many years. The opinion was expressed that the total value of the small bridges built is more than that of the million dollar bridges.

15. The actual energy of the blow of the hammer as delivered to the pile in making tests is a vital matter when determining pile values by formula. The weight and fall of the hammer are easily determined, but the friction losses are more difficult.

16. The Vibropile used in Europe to some extent was discussed, but it was brought out that its use in the United States where it was introduced some years ago was negligible.

17. Pile Foundations. The material below the pile points is of decisive influence on the whole thing. There are lots of things that must be considered in determining how much load to put on the foundation other than the descriptive nature of the soil and mere weight. The foundation has to be designed and determined by what is below, how far the soft strata extend and the neighboring conditions. The mere presence of a building alongside may alter the bearing power of your own foundation. The case is not a simple one of assigning a load value whether by a loading test or by description of material. There is the factor of the permissible amount of settlement, whether it is 1/8 of an inch or 1 1/4 inches, all of which enter into the question of how much one pile will carry or how much the entire group of 120 piles will carry.

18. In other words, it is obvious on the face of it that to seek a formula or to hope to find a formula is a mistaken effort.

19. The Engineering News Formula. Sometime in the eighties Arthur M. Wellington, when he found that a great many engineers were asking, "Well, how much will a pile carry?" reached back to energy formula methods, which were not at all new, and devised and wrote out a simple practical formula with a large safety factor designed purely for energy conditions. In the years which followed I have seen many engineers inquiring, "Why don't these formulae work in my case?" Usually it was this so-called Engineering News Formula or the Wellington Formula. "It shows twenty-eight tons and I only got fourteen", or something of that sort.

20. It turned out that these people were driving in clay or river mud, or something of that sort. Now, the mere idea that an engineer can take an energy formula and apply it to the mud in the river bottom is rather absurd. But it is largely, I think, because of that sort of thing, the abuse of an energy formula, that this condition came about which Mr. White and Dr. Terzaghi have characterized by saying that this formula has lost currency. I hope that all formulae for piles will have lost currency soon, and that pile designs and pile calculations will be put on some other basis. But to overlook the fact that in such circumstances as a fairly compact and fairly uniformly dense granular stratum with no softer layers or strata lying below you can get dependable results with an energy formula, as in Mr. Mason's case, would be equally wrong.

21. That is a limited number of cases, and I suppose that the total number of cases in which piles are driven in sand is not a very large fraction of the total. Another member estimated 50% of the total is in sand.

22. I am thinking of the large amount of work done in material that is called river mud. Let us look aside from clay. Let us waive the question of whether anybody ever ought to drive piles in clay. Take the case of alluvial ground where a great many piles are driven. It seems so obvious that none of the normal methods of calculation, certainly nothing like an energy formula, should be applied there, that I am wondering why there has been so much discussion about it. One gentleman said: "I hope we can devise a formula that will give us the bearing value of a single pile. It looks to me as if that is, on the very face of it, impossible.

23. Mr. Crandall made a statement this morning on piles, which seemed to me to cover in brief form just about all that needs to be said on the subject in the way of limiting the proper scope of anything tangible in the way of a formula such as an energy formula. A formula, of course, that is based on driving must be an energy formula. It cannot be anything else. If we are looking for a pile driving formula, one that is based on what happens during driving, it can hardly be anything else than an energy formula. Then you have merely to pick out of the vast amount of literature on energy formulae on pile bearing value what you choose to be the safest and at the same time the most economical.

24. The so-called Engineering News Formula in my recollection has never been indicted on the score of being wasteful. Nobody has ever kicked about it giving too much safety. If there has been any kick it has been because, as in Mr. Mason's case, the safety limit beyond the design loading was not ample.

25. A member stated he was of the contrary opinion. Criticism has been the other way. I do not want our guests to go away with the idea that the Engineering News Formula has completely lost caste in the United States, because as far as building foundations are concerned I should say that at least ninety per cent of the specifications that are issued by all authorities in the United States refer to the Engineering News Formula today.

26. Another member did not agree to this.

27. A member continued and stated that in connection with the work that we have been doing--these are friction piles, not piles to rock--at least ninety per cent of the specifications apply to that, because I do not know of any other formula or any other method as to pile driving that is used.

28. It was brought out that New York City has an arbitrary measure right now that wood piles are to carry so much load and the friction piles so much load, and it is not based on the Engineering News Formula at all. It was further stated, however, that the Engineering News Formula is involved and that the Code gives a certain maximum limit and that the depth to which you must drive to obtain that limit is a function of the Engineering News Formula.

29. Two absolutely true things are brought out--that in pile foundations any formulae cannot take into consideration soft materials below the point of the piles; also the question of large groups of piles as against individual piles. But from our experience of thirty years approximately with the Engineering News Formula we know that there is sufficient safety factor in that formula so that reasonably sized groups are satisfactory with the loads that we use.

30. A member stated in effect that as long as you get into it you have to use it. They have insisted on using it and your building codes in the United States apply equally to sand and to clays. In New York City they apply the Engineering News Formula in their building inspection to clays, to river muds, to silts, to what not. There is no distinction, so far as I know, in any code in North America.

31. To this a reply was made: That indicts the engineer rather than the formula. Isn't the thing that we need here a certain amount of engineering sense--basically, I mean. There are places where piles should not be used and they have been used in such places in the past. No formula, no codification of values, and no settlement measures are going to help you over that. That is just poor engineering. It seems to me that first of all you have to consider the field and the service of piles, and the great many different conditions under which they serve, and fit the considerations to that.

32. A member stated: I was going to say that I have made a few tests--about forty. From those tests and from the driving records that I have I want to bring out the fact that, in general, the Engineering News Formula was too conservative. We were using too many piles for the loads we had to carry. That was the conclusion I came to. As far as Engineering News Formula is concerned there are just a lot of cases where I got a load of ten times as much as the formula showed and in some cases I couldn't move a pile. In a good many instances I put as high as one hundred and seventy tons on an eight inch pile and brought it up to its elastic level. That was in gravel. In a similar condition I couldn't move the pile. I pulled up one of the anchor piles and it took seventy-five tons to pull it up. I have had a lot of tests similar to that. At one time I petitioned the District Engineer of the U.S.B.P.R., under whom we function and whose money we spend, to modify that Engineering News Formula in some way, especially in places where we had gravel and I knew that we could not move them. I suggested that we use a factor of three.

33. Another member stated: Our standard practice is to drive to one loading capacity, that is, thirty tons. Also our practice is to drive to fifty per cent over that, as determined by the Engineering News Formula. Thirty tons with the No. 1 Vulcan hammer, which is the universal tool today, is two and a half blows to the last inch. We drive to four blows which is about forty-three tons, and we have found that that is safe, irrespective of the grouping of the piles, except due to underlying stratum or peculiar conditions such as Dr. Terzaghi has mentioned. We don't often test over four piles in a group, because two hundred and forty tons or sixty tons to the pile is quite a load test.

34. In an effort to summarize the situation as to the formula it was stated: I would like to say one thing on the Engineering News Formula, looking at it from a slightly different light. We are looking at it tonight from the standpoint of all this very considerable knowledge that has been gathered in the last few years in regard to foundations. But there are a great many engineers still practicing in this country who have not had the benefit of that knowledge. I suppose the vast majority of them have not. The Engineering News Formula has been a device that an engineer could use in his specifications in order to make definite some means of determining what he called the bearing value of the pile. He was wrong in a great many cases. He was right in some cases as to arriving at a safe structure. But it was, and still is a convenient means for an engineer to put something definite into a specification.

35. Now how many times have buildings failed where the Engineering News Formula was used. With the vast foundations and the very ticklish foundation work that is done today, of course the Engineering News Formula would not be justified, but I believe that a great many engineers, in the present state of knowledge of the engineering profession at large, are still forced to use the Engineering News Formula.

36. I am merely trying to state a fact, a condition, and not a justification at all. If you take engineers in some remote cities, or even in large cities, who are grinding out work, a great many of them don't have time to study, or possibly the ability or the inclination to study these modern developments. He is forced to do something and he falls back on that.

37. It was then stated by a member that you make foundations sometimes where you don't need an engineer. They can be done by rule of thumb.

38. The summary continued: I don't think that should be done. I merely am trying to bring out that the average practicing engineer of this country was practically forced to use that formula. He didn't know any better, dependent on the state of his knowledge. It was a convenient device to put into a specification.

39. It is the duty of the Committee to broadcast the more recent knowledge in regard to soil mechanics and bearing value of pile foundations with everything that has been brought out tonight with regard to the importance of the strata below, in regard to the group value not being merely the product of the number of piles and the individual pile value.

40. When you burn down the house of the Engineering News Formula you must have another house for the patient to live in, because you face a practical situation. We have to give him something better.

<u>41. The Hiley formula</u> for determining the bearing value of a pile: This formula was mentioned but not discussed as to its derivation. While the Engineering News formula has such a wide margin for safety that it works, it is probably too conservative for results. One member suggested that a great number of pile records where the piles were loaded to failure--that is, the essential condition--should be taken and then the bearing value by the Engineering News Formula and by the Hiley Formula should be computed. However, in this connection no confidence was expressed in formulae as a safe means of universal determination of the safe load for piles.

42. <u>Elastic Behavior of Pile</u>. The elastic behavior of the pile is recognized in Shanghai and in Göteborg, Sweden. The pile sinks under the effect of the hammer blow but it rebounds and remains stationery at some depth below the starting point. There is thus the total penetration, the rebound and the permanent penetration--the rebound plus permanent penetration equal the total penetration. The rebound is due to the elastic reaction of the pile (or of the soil or both).

43. <u>Elastic Behavior of Pile (2)</u>. Observation at Rotterdam showed that a pile penetrated quite easily and a neighboring pile penetrated heavily. Loading tests showed both piles had precisely the same penetration curve at loading and the same curve for pulling.

44. Also in the Conference Proceedings are recorded experiences by the Dutch in driving a couple of comparative pile tests. One pile needed about 30 blows per decimeter and the other needed eight. Both piles were loaded and gave precisely the same loading and pulling curves. This duplicated the above experience.

45. The first case was in about 16 meters of mud, silt, and peat, with a skin friction of about 1 1/2 T. per square meter and then into sand. In the second case they drove through clay with a skin friction of four tons per square meter and then also into sand. The same type of soil in both cases.

46. <u>Elastic Behavior (3)</u>. Observations with a steam hammer at Dutch tests showed that permanent penetration was about half the total.

47. A member stated that with a steam hammer at 60 blows to the minute there is a slight rebound.

48. Dr. Tersaghi remarked: "It is very different for different soils".

49. A case was mentioned in which the rebound was equal to the total penetration (permanent penetration zero). In that case the hammer did not apply enough force to overcome the elastic limit.

50. It was suggested that by use of a drop hammer h_o, the value of the elastic property may be determined by finding the drop which just fails to produce a permanent penetration--providing the line which gives the relation between the penetration and the fall of the hammer is straight.

51. <u>Elastic Behavior (4)</u>. Piles, for instance in Holland where the points are in the sand, you can go on loading and loading and you get deeper and deeper penetration but anything well defined is never reached. There is no elastic limit to it--no definite yield point. In that case one cannot possibly do anything else except specify an arbitrary limit, for instance one or two inches and define it by that. This would be an arbitrary standard and has absolutely nothing to do with safe bearing value.

52. A member stated that would be a high limit in the City of Mexico. If it settled a foot that would be a very good foundation. Another member stated that in Mexico City single piles behave just as decently as they do in other countries. The trouble starts when they appear in families.

53. <u>Formulae and Pile Driving Tests and Loading Tests</u>. Formulae attempt to give the value of a pile. Innumerable tests in underpinning show a value of a pile of from 5 to 40 tons in sand if the penetration is controlled.

54. In Denmark records taken in the evening will show one result, and if you take the same records the next morning just after starting the driving you will perhaps get only one-tenth what you had the previous day.

55. This is a safe indication that the pile driving formulae is absolutely useless if you get an appreciable difference between intermissions.

56. In Denmark when it is clay bottom we practically always have much less penetration after waiting some hours. In one case a heavy concrete pile was driven 25 meters. The formula gave 40 tons capacity by test blows in the afternoon. Twenty-four hours later the hammer test and formula showed 400 tons capacity. The loading test gave twenty-seven tons.

57. A member stated that if you have a good foreman and a block of wood you can get any value you want.

58. The pile driving formula at best takes care of only one factor and that is an attempt to determine the bearing value of a single pile. We still have the influence of the underlying soils of softer consistencies below the point of the pile, and the effect of groups of piles.

59. In Vienna piles are driven into a stratum of sand and gravel fairly well compacted and going down to rock. Records of loading tests on many individual piles in the entire foundation show that the ratio between the settlement of the individual pile and that of the entire pile group ranges between five times, which is the more favorable case, to twenty or thirty times the subsidence of the individual pile. The pile loads as a rule are 20 to 25 tons per pile for conical piles. The individual pile shows a settlement of a fraction of a millimeter and the entire foundation between one centimeter and two centimeters.

60. The engineer is interested in the fundamental thing, which is not what the individual pile will do but what the whole foundation will do. That makes it immediately a general problem of soil mechanics, where you have to consider everything.

61. The Vienna experience indicates that with known bearing value of individual piles as determined by loading tests the entire foundation for the average building settles a given amount. This ratio between individual load tests and building settlement has been found to be fairly constant. It would appear that empirical rules for each city or for each set of uniform conditions may be established for use in building pile foundations. This pile driving and foundation matter may result in

empirical practices in different communities where the soils have characteristics that vary between definite and rather close limits.

62. The opinion was expressed that it has largely gone that way and also that there is a tremendous number of piles wasted by putting them too close together, where a smaller number would do even better work.

63. Standard practice of one of the companies is to drive to 30 tons loading capacity or 50% over that by the Engineering News Formula. Thirty tons with the No. 1 Vulcan hammer is 2 1/2 blows to the last inch. They use 4 blows or 43 tons to the pile. It is stated that this has been found safe irrespective of the grouping of the piles except due to underlying strata or peculiar conditions. As many as 4 piles to a group are tested with a two hundred and forty ton load. Settlement is in the neighborhood of 1/4 inch.

64. The criterion for the validity of any formula regardless of how accurate or scientific it may be, is that the dynamic pile driving resistance is identical with the static resistance. In some cases they are identical--such as in permeable material consisting of sand, gravel, fills without any cohesion. Then the Engineering News Formula has such a wide margin of safety that it works, but it is probably too conservative for results. The Hiley Formula is better.

65. Static formulae are absolutely worthless as every static formula, without exception, assumes that the friction along the circumference of the pile increases with the depth. Now all piling records which have come to my attention including small scale tests show that in cohesionless sand if that sand is dense you don't get the pile in at all without jetting and if the sand is loose the skin friction is independent of the depth.

66. There is still a hope that a practicable workable formula may be devised but it will only apply to the single pile and can take no account of underlying strata overloaded by the grouping of piles as a whole.

67. Shape of Pile Foundation. On the Pacific Coast a factory with a length of about 1000 feet was built on a pile foundation with piles 60 to 80 feet long with extremely conservative loading between 10 and 15 tons per pile. Care was exercised and all structures in the vicinity were first investigated. Near by a railroad bridge had been built 40 years before on piers with piles 50 feet long. It had not settled appreciably. The bridge piers were narrow compared with the depths of the piles, and therefore the load of the bridge was spread over a wide area. One hundred feet below was a layer of clay. The narrow bridge piers exerted practically no pressure thereon. The factory covering a large area gave almost full load of 2 tons per square foot on the clay. The factory settlements were more than a foot and a few inches, extremely detrimental to the machinery.

68. Foundation in loose sand about 50 feet thick above an inclined rock bottom was to be built on concrete piles but the ground water contained 40 milligrams CO_2 per liter of water, dangerous to the pile. The sand was transformed from the loose to the dense state by using an ingenious method for driving masses of sand, gravel and trass in the form of piles into the mass. The sand was compacted and the surface actually settled one foot. The foundations could then be built direct on the sand.

69. Proposed Manual for Pile Driving. The opinion was that the time had not yet come for preparation of a Manual. Sufficient information is not yet on hand.

70. The Manual will not be a set of rules for designing a pile foundation, but it will state certain principles, call attention to various possibilities and eliminate the dangerous feeling of certainty. It will be for the use only of engineers of sound good judgment, good basic training and experience. It is not intended to be of the specification type. It will rather state principles and cardinal control items. It should contain a glossary and definition of terms.

71. Future progress must consist in the further collection of data, particularly of failures of pile foundations, definitions and of the material assigned to individual members.

72. It is hoped before the Annual Meeting to outline certain principles so far as we have gone.

SUGGESTED PROCEDURE FOR INVESTIGATION AND COMMITTEE STUDY FOR ITEMS 10, 12 and 13

X Item 10--Long time factors in bearing values of the piles.

A. Altered conditions
   1. Change in ground water level
   2. Change in lateral stability due to adjoining excavations above the supporting media.
   3. Loss of pile support due to adjoining pile support within or below the supporting media.
   4. Change in lateral stability due to placing of adjacent fills or surcharges.
   5. Overloading of piles through friction induced by adjacent fills or surcharges.

B. Consolidations
   1. Of the supporting media
   2. Of materials below supporting media
   3. Resulting from adjacent fills or surcharges
   4. Influence of remoulding on consolidation of soils above the pile points

C. Corrosion

<u>Item 12</u>--Preliminary investigation for design of the pile foundations.

A. Geological
1. Available borings records
2. Geological history
3. Ground water surveys
4. Borings
(a) Dry sample borings including undisturbed samples
(b) Wash borings
(c) Probings

B. Test Piles
1. Driving tests
2. Loading tests
3. Driving records in locality through similar materials

C. Existing Structures
1. Records of driving and loading and of pile settlements and foundations of structures in same locality through similar materials.
2. Local practices
3. Records of structures founded on similar soils but in other localities.

<u>Item 13</u>--Elements in the design of a Pile Foundation.

A. Manner in which the pile will derive its support.
1. Through friction
2. Through point bearing
3. A combination of friction and point bearing

B. Lateral stability derived from surrounding media.
1. To prevent buckling action
2. In supporting lateral forces

C. Strength of piles as structural elements.
1. Strength as a column for support of vertical loads
2. Strength as a beam for support of lateral loads
3. Strength of pile against dynamic resistance in driving
4. Influence of out-of-plumbness for bent on the strength of the pile and tolerances permitted by the design.

D. Durability of the Piles
1. As influenced by ground water
2. As influenced by soil acids or alkalis
3. As influenced by electrolysis; either electrolytic or self-induced.

E. Loads
1. Dead
2. Live
3. Wind
4. Lateral forces

F. Limitations imposed by supported structure
1. Total settlement as influenced by connecting elements (sewers, water lines, etc.)
2. Differential settlements as limited by strength and flexibility of the structure.

Above submitted 24 June, 1936, by Mr. Carlton S. Proctor, Member of Committee.

No. H-19

COMMENTS ON VARIOUS PAPERS
(Editorial notes abstracted from oral and written communications.)

<u>Paper I-5</u>: In a communication the fact was pointed out that in the computation of the skin friction of such closely spaced piles one must not assume the total surface area of the piles, but of the smallest circumferential area of the entire body along which failure would actually take place. For the case described in this paper the friction value will be increased from 1.63 to 2.2 tons per sq m.