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PAPERS AND DISCUSSIONS

OF GENERAL INTEREST

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A CONSIDERATION OF PILE DRIVING WITH APPLICATION OF PILE LOADING FORMULA

By Robert D. Chellis*

(Presented at Meeting of Designers' Section, Boston Society of Civil Engineers, March 13, 1940)

The broad question of whether or not piles are required under a structure is a problem in soil mechanics. In this connection knowledge of the site obtained from adequate borings is indispensible, and it often proves to be poor engineering judgment and false economy to skimp on the number, depth and quality of the borings. Soil samples, preferably in as undisturbed a state as possible, should be carefully inspected. This paper will not enter into a presentation or discussion of methods for determining the need of piles, but is concerned only with the driving of the piles, based on the assumption that their use has been found advisable. Although considerable of the data presented are available elsewhere in various widely scattered sources, they are neither readily available nor presented in convenient form. It is the object of this article to make a hitherto complicated formula workable for the average field man, and to assemble and correlate in one convenient source all of the necessary data and formulas to enable one to handle any normal pile driving problem quickly and easily, with a degree of accuracy adequate for all practicable purposes, and with uniformity of results.

Briefly, piles are effective in transferring the load as columns through soft upper strata to a hard substratum; in transferring the load through soft upper strata deep into stiffer strata below which

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are considered to have adequate distributing value; and sometimes in compacting the soil.

It is recognized that experience and judgment are of great value in pile driving operations, and it is considered that any pile driving formula should be looked upon as a measure of the hardness of the soil and in that way an aid to judgment. The application of the theory and data presented have been checked against actual driving operations whenever opportunity arose. The use of the proposed procedure is intended to promote four principal objects, namely: uniformity of driving results with different kinds of piles and types of hammers; economies of pile lengths due to such uniformities by avoiding excessive driving in some cases; avoidance of failure to drive the piles to an adequate depth into the bearing strata to obtain proper permanent safe bearing values on the soil; and a guard against obtaining apparent friction values, in the case of friction load-carrying or uplift piles, which are in excess of the actual values.

The leading pile formula which has been in most general use in the past in this country is the so-called *Engineering-News*[1] formula, while Redtenbacher's formula has been used quite extensively in Europe. No attempt will be made herein to give a history and comparison of the various formulas in use in this country and abroad, such as the Eytelwein, Navy, Dutch, Redtenbacher, Rankine and Benabenq formulas, as this has been done by various writers.[2,3,4] The *Engineering-News* formula has the advantage of simplicity, but in the interests of this simplicity neglects entirely many widely variable factors and uses fixed coefficients for other variables. The factor of safety actually obtained by the use of this formula may vary widely, in many cases being much greater than necessary, due to the attempt to provide a very short formula which will cover a wide range of conditions with a factor of safety adequate to fit the worst cases. By comparison with the following formula, the factor of safety obtained by use of the *Engineering-News* formula may vary from as low as ½ to as high as 16 or more. Unduly high capacities are indicated with small penetrations in particular. It would be desirable

[1] A. M. Wellington, "Engineering-News", vol. XX, pg. 510-12, 1888.
[2] J. Stuart Crandall, "Piles and Pile Foundations", Journal B. S. C. E., vol. XVIII, No. 5, May 1931.
[3] "Concrete Piles", published by Portland Cement Association.
[4] "The Science of Foundations—Its Present and Future" by Charles Terzaghi, paper No. 1704, Transactions, A. S. C. E. 1929.

to design piling to a more uniform factor of safety and to a more uniform length, if this could be done in a simple manner.

This article is based on the work of Hiley,[5] Redtenbacher[6] and Terzaghi[7] who have developed dynamic pile loading formulas to include all or most of the energy losses. These formulas, and other modifications, have been coming recently into more frequent use in this country. It is the object of this paper to summarize these results in a comprehensive formula, with tabulations of the various weights and dimensions of parts of pile drivers and piles and of the several coefficients required in the formula, so that all necessary data will be available in this one source, to enable the engineer in the office, by entering these values in the formula, to produce a simple formula of the *Engineering-News* type for uses in the field, or, if desired, to specify merely the amount of the final penetration per blow desired. This simplified formula will contain only two coefficients, determined by, and constant for, the particular conditions of the job, such as size and type of hammer, length and material of pile, type of driving head, and driving conditions, instead of empirical coefficients which attempt to embrace the entire range of conditions by the use of a factor of safety large enough to cover the worst ones. Such a reduced formula will apply only to the particular set of conditions obtaining on the one job, but will hold throughout the job as long as no essential change takes place in them. Inspection of formulas (1a) and (1b) will show that they will resolve themselves into the familiar *Engineering-News* type when a factor of safety has been chosen and numerical values assigned to the various terms, to form constant coefficients for the job, the value of the penetration "s" remaining the only variable.

Dynamic Formula

The theory of dynamic impact, upon which the following formula, and nearly all other formulas in general use up to the present time, are based, namely that the ultimate carrying capacity is the same as the ultimate driving resistance, results in a so-called "dynamic" pile-driving formula. Such a formula can apply only in the case of cohesionless strata, such as sand, gravel or permeable fill, in which cases

[5]"Pile Driving Calculations with Notes on Driving Forces and Ground Resistance", by A. Hiley, "The Structural Engineer", vol. VIII, July and August 1930.
[6]"Prinzipien der Mechanik und des Maschinenbaues", F. Redtenbacher.
[7]*Supra*, note 4.

the resistances acting while the pile is being driven are practically the same as those acting on a pile carrying a static load. In the case of driving piles in plastic material such as soft clay or fine grained silt, there is no relation between the temporary resistance to driving and the permanent resistance to the applied load on the pile. In materials of this second class the friction action during driving is very much less than the amount which occurs after a period of time, but the resistance to a dynamic blow is far greater than the resistance to a static load carried over a long period. It is an error sometimes made to use a pile driving formula with materials of the second class. There are many types of soils which partake of the characteristics of both classes, and in such cases judgment should be used as to the extent to which dependence may be placed on a formula of this type.

The *ultimate* carrying capacity (R_u) (considered as ultimate resistance to driving) of each pile would be obtained from the relation $R_u s = W_r h$ if it were not for energy losses due to various sources. The following formula, which considers energy losses due to the principal causes (efficiency of hammer, impact, temporary compression in pile cap and head, temporary compression in pile, temporary compression or quake of ground) will give the *ultimate* carrying capacity (R_u):

Formula for Use with Drop Hammers and Single-Acting Steam Hammers:

$$R_u = \frac{e_f W_r h}{s + (1/2)(C_1 + C_2 + C_3)} \times \frac{W_r + e^2 W_p}{W_r + W_p} \tag{1a}$$

Formula for Use with Double-Acting and Differential-Acting Steam Hammers:

$$R_u = \frac{12 e_f E_n}{s + (1/2)(C_1 + C_2 + C_3)} \times \frac{W_r + e^2 W_p}{W_r + W_p} \tag{1b}$$

for Case of $W_r > W_p e$ which is the usual case and in which conditions favorable to driving prevail

In which R_u = *ultimate* carrying capacity of pile (considered as ultimate resistance to driving), in pounds, before applying any factor of safety.

W_r = weight of ram in pounds (see Table IV).

W_c = weight of casing in pounds, for double-acting and differential-acting steam hammers (see Table IV). At maximum rated energies for Vulcan-California and differential-acting hammers $E_n = (W_r + W_c)\ (h/12)$.

E_n = rated energy of hammer per blow, in foot-pounds, as published by manufacturers, and listed in Table IV for various numbers of

strokes per minute. The hammers should run at the maximum speeds listed, if possible, to obtain the greatest overall efficiency. (In the case of Vulcan-California and Super-Vulcan differential acting steam hammers this is taken by the manufacturer as the sum of the energies obtained by multiplying the weight of the ram by the stroke plus the product of the piston area times the published steam pressure at the hammer, with the maximum value equal to the sum of the weights of the ram plus casing times the stroke, $E_n = (W_r + W_c)\ (h/12)$. This is based on the theory that the steam pressure can not exceed the weight of the casing without causing the casing to lift from the pile). (In this case of McKiernan-Terry double-acting hammers the value of E_n is based on indicator diagram readings, confirmed by high speed moving picture readings of the velocity of the ram at impact, and while these values agree reasonably well with the theory of limiting the steam pressure by the weight of the casing, the variations are enough so that it is recommended that the rated energies be used, since the hammers act most efficiently at these figures).

$h =$ height of free-fall of ram in inches for drop hammers; normal (shortest) stroke of ram in inches for single-acting steam hammers; and normal stroke of ram for double-acting and differential-acting steam hammers. (See Table IV).

$e_f =$ efficiency. The following percentages are suggested:

100% for drop hammers released by trigger;

75% for drop hammers actuated by rope and friction winch, but bearing in mind that this figure may decrease when the drop is small or the drag considerable, and increase somewhat if the drop is very large or the drag not great;

(w) 75% for single-acting steam hammers;

(w) 65% for Vulcan-California double-acting hammers;

(w) 85% for McKiernan-Terry double-acting hammers. Since the rated energies of these hammers are based on indicator diagrams, losses due to back pressure, preadmission of steam before completion of the downstroke, expansion losses due to drop from entering pressure of steam, mean effective pressure in the cylinder, wire drawing of steam, and losses on valves and ports have been deducted before obtaining the rated energies, leaving only mechanical losses such as those due to piston ring friction and tight packing to be covered by the value of "e_f". (The manufacturer recommends 90% as the lowest value of e_f for these hammers, even when operated under unfavorable circumstances);

(w) Includes 10% to cover poor condition of hammer, wear, improper adjustment of valve gear, poor lubrication, unusual weather conditions causing condensation, unusually long hose, restricted areas at hose connections, minor hose leaks, unduly tight packing in types of hammers having manual adjustment of packing, un-noted minor drops in steam pressure, which will reduce stroke, binding in guides, etc.

(w) 75% for differential-acting steam hammers. Since the rated energies for these hammers are based on the product of the entering steam pressure times the areas of the piston, the value of e_f should be such as to cover losses due to wire drawing of steam, piston ring friction, back pressure resulting from preadmission of steam just prior to impact, losses in valves and ports, tight packing and other mechanical losses. It is claimed that the nonexpansive use of steam in the steam cycle in this type of hammer obviates a drop from the entering steam pressure to mean effective pressure. (The manufacturer recommends 84% for the value of e_f for the large and medium size hammers, and 80% for the small sizes, for hammers in first-class condition and operated under favorable circumstances).

W_p = weight of pile in pounds, including shoe and driving cap for drop hammers and single-acting steam hammers; weight of pile including shoe and weight of anvil in case of double-acting and differential-acting steam hammers (see Tables IV and V). Note that steel sheet piles are sometimes driven in pairs, and the weight of both should be included.

l = length of pile, in inches.

L = length of pile, in feet.

e† = coefficient of restitution:

= 0.5 for ram of double-acting hammers striking on steel anvil and driving steel piles or precast concrete piles;

= 0.4 for ram of double-acting hammers striking steel anvil and driving timber piles, also for striking steel helmet containing wood and driving steel piles;

= 0.4 for ram of single-acting or drop hammers striking directly on head of precast concrete piles not fitted with driving cap;

= 0.32 for ram of single-acting hammers striking on steel plate cover of wood cap of steel piles;

= 0.25 for ram of single-acting or drop hammers striking on well-conditioned wood cap of driving cap in driving precast concrete piles or directly on head of timber pile;

= 0.0 for a deteriorated condition of head of timber pile or of wood cap and for excess packing in driving cap.

s = final penetration of pile in inches per blow (using average of last five blows for drop hammers and of last three inches for other types).

C_1 = temporary compression allowance for pile head and cap. (See Table I).

C_2 = temporary compression of pile. (See Table II).

C_3 = temporary compression allowance for ground for average cases where pile is driven in penetrable ground. (See Table III).

A = average area of cross section of pile in square inches (shell only, in case of steel pipe pile, and average area of mandrel obtained from total weight and dimensions in case of Raymond piles). (See Table V).

†Values of "e" from Hiley.

E = modulus of elasticity for pile material.

p_1 = stress per sq. in. on pile head or cap. $\left(p_1 = \frac{R_u}{\text{area of pile head}}\right)$

p_2 = stress per sq. in. on average cross section of wood or concrete piles, or shell area of tubular steel pile shells, or on area of steel sheet piling, or on average area of Raymond mandrel.

$\left(p_2 = \frac{R_u}{A}\right)$

p_3 = stress per sq. in. on horizontal projection of pile tip, including driving points under steel pipe casings. $\left(p_3 = \frac{R_u}{\text{area of tip}}\right)$

NOTE: All of the above percentages have intentionally been given as slightly on the low side for all types of hammers, compared to the values which might be used provided the hammers were all in excellent condition, in order to be reasonably certain of obtaining at all times approximately the intended energy as a minimum, since possible slight overdriving is preferable to underdriving. However, it is very likely that there are hammers in use today which do not deliver more than 50%-60% of their rated energies.

In so far as use of the formula is concerned, it should be noted that the driving of concrete filled pipe piles, Union pile shells, and the steel mandrels for Raymond piles consists of the driving of steel and not concrete, as the concrete is poured later as a filling in the pipe or casing.

FIELD FORMULA

In the usual development of either formula (1a) or (1b) into a formula for field use the value of R_u is known, being the product of the safe pile load chosen and the factor of safety selected, the values of C_1, C_2 and C_3 are taken from Tables I, II and III, and the formula solved for the desired penetration "s".

The formula for field use will then take the form $R = \frac{X}{s+x}$ (2)

In which R = safe design load in tons (due to dynamic resistance) to be assumed.

X = numerical value for which formula is to be solved.

x = term $(1/2)(C_1+C_2+C_3)$.

s = penetration obtained by solving either formula (1a) or (1b) for "s".

TABLE I*

TEMPORARY COMPRESSION ALLOWANCE C_1 FOR PILE HEAD AND CAP

Material to Which Blow is Applied	Easy Driving p_1 = 500 Lb. Per Sq. In. on Head or Cap	Medium Driving p_1 = 1000 Lb. Per Sq. In. on Head or Cap	Hard Driving p_1 = 1500 Lb. Per Sq. In. on Head or Cap	Very Hard Driving p_1 = 2000 Lb. Per Sq. In. on Head or Cap
Head of timber pile	.05″	.10″	.15″	.20″
3″ to 4″ packing inside cap on head of precast concrete pile	.05″ to .07″	.10″ to .15″	.15″ to .22″	.20″ to .30″
½″ to 1″ mat pad only on head of precast concrete pile	.025″	.05″	.075″	.10″
Steel covered cap, containing wood packing, for steel piling or pipe	.04″	.08″	.12″	.16″
3/16″ red electrical fibre disc between two ⅜″ steel plates, for use with severe driving on Union pile	.02″	.04″	.06″	.08″
Head of steel piling or pipe	0	0	0	0

TABLE II

TEMPORARY COMPRESSION VALUES OF C_2 FOR PILES

Type of Pile	p_2 = 500 Lb. Per Sq. In. for Wood or Concrete Piles; 7500 Lb. Per Sq. In. for Steel, Net Section	p_2 = 1000 Lb. Per Sq. In. for Wood or Concrete Piles; 15000 Lb. Per Sq. In. for Steel, Net Section	p_2 = 1500 Lb. Per Sq. In. for Wood or Concrete Piles; 22500 Lb. Per Sq. In. for Steel, Net Section	p_2 = 2000 Lb. Per Sq. In. for Wood or Concrete Piles; 30000 Lb. Per Sq. In. for Steel, Net Section
Timber pile (E = 1,500,000)	.004″ x L	.008″ x L	.012″ x L	.016″ x L
Precast concrete pile (E = 3,000,000)	.002″ x L	.004″ x L	.006″ x L	.008″ x L
Steel sheet piling, Simplex tube, pipe pile, Union Monotube shell, Raymond steel mandrel (E = 30,000,000)	.003″ x L	.006″ x L	.009″ x L	.012″ x L

Note: All other values in direct proportion to E and p_2.
When computing p_2 for a Raymond steel mandrel, it is suggested that the weight of the mandrel be divided by 3.4 x lg. of pile in feet to obtain the average area.

TABLE III*

TEMPORARY COMPRESSION OR QUAKE OF GROUND ALLOWANCE C_3

	p_3 = 500 Lb. Per Sq. In. on Pile Tips or Driving Points	p_3 = 1000 Lb. Per Sq. In. on Pile Tips or Driving Points	p_3 = 1500 Lb. Per Sq. In. on Pile Tips or Driving Points	p_3 = 2000 Lb. Per Sq. In. on Pile Tips or Driving Points
For vertical sided piles**	0 to .10″	.10″ (to .20″)	.10″ (to .30″)	.05″ (to .20″)

*Tables I and III largely from "Pile Driving Calculations with Notes on Driving Forces and Ground Resistance" by A. Hiley, printed in *The Structural Engineer,* vol. VIII, July and August 1930. For a fuller discussion of the means of obtaining these values see this reference. For the purpose of this article the values represent average conditions and may be used.

**It is recognized that these values should probably be increased in the case of piles with battered faces, but no test data are available at the present time to cover this condition.

TABLE IV

WEIGHTS OF VARIOUS PARTS OF HAMMERS AND DRIVING CAPS IN POUNDS, WITH STROKES AND ENERGIES

Type / Item	Vulcan Drop Hammers						
	K	*M*	*P*	*S*	*T*	*V*	*W*
Ram (W_r)	500 600	700 800	1000 1200	— 1500	— 1800	2000 2500	— 3000
Helmet for steel sheet pile	—	—	—	—	600	700	750
Pile cap	280	300	520	720	760	900	970
Sheeting cap	Vary considerably, but about same weight as pile caps.						

Type / Item	Warrington-Vulcan Single-Acting Steam Hammers						McKiernan-Terry Single-Acting Steam Hammers		
	OR	0	1	2	3	4	Closed	Closed	Closed
Shortest stroke (h) (at zero penetration)	39″	39″	36″	29″	24″	21″	36″	48″	32″
Ram (W_r)	9300	7500	5000	3000	1800	550	7500	7500	14000
McDermid plate	—	—	65	57	—	—	—	—	—
Dished cap	—	—	60	50	28	—	—	—	—
Helmet for steel sheet pile	—	1400	750	700	600	—	—	—	—
Pipe cap	—	—	750	650	—	—	—	—	—
Anvil block for *H* piles	—	—	—	—	—	—	2200	2200	3400
Anvil block for 24″ conc. piles	—	—	—	—	—	—	3000	3000	—
Anvil block for 18″ conc. piles	—	—	—	—	—	—	2800	2800	—
Cup anvil	—	—	—	—	—	—	2600	2600	—
Raymond cap block (2″ plate, 3″ wood, 1″ plate)	—	—	225	—	—	—	—	—	—
Driving head assembly for use with Union piles	650	650	650	650	650	650	650	650	650

Small bathtub Raymond follower approximately 2′ lg. = 800 lb. (large ones extend to 8′-10′ lg., with weights up to 4000 lb.)

TABLE IV (*continued*)

WEIGHTS OF VARIOUS PARTS OF HAMMERS AND DRIVING CAPS IN POUNDS, WITH STROKES AND ENERGIES

Type	Super-Vulcan Differential-Acting Steam Hammer Open Type					
Item	18*C*	30*C*	50*C*	80*C*	140*C*	200*C*
Normal stroke (h)	10½″	12½″	15½″	16½″	15½″	15½″
Ram (W_r)	1800	3000	5000	8000	14000	20000
Casing with standard base (W_c)	2339	4036	6782	9885	13984	19050
Casing with McDermid base (W_c)	2364	4090	6845	10013	—	—
Mfr's rated energy, ft. lb. per blow (E_n)	3600	7260	15100	24450	36000	50200
Strokes per minute	150	133	120	111	103	98
Standard plate	15	40	48	75		
McDermid plate	22	55	65	80	—	—
Dished cap plate	30	55	70	125		
Driving head assembly for use with Union piles	650	650	650	650	650	650

Type	Super-Vulcan Differential-Acting Steam Hammer Closed Type							
Item	600	1100	1800	3000	5000	8000	14000	20000
Normal stroke (h)	7½″	9″	10½″	12½″	15½″	16½″	15½″	15½″
Ram (W_r)	600	1100	1800	3000	5000	8000	14000	20000
Casing (W_c)	1232	1839	2339	4036	6782	9885	13984	19050
Mfr's rated energy, ft. lb. per blow (E_n)	1125	2180	3600	7260	15100	24450	36000	50200
Strokes per minute	225	181	150	133	120	111	103	98
Flat anvil	—	—	200	392	588	877	1460	1450
Cup or bell anvil	55	101	135	214	358	592	—	—
Driving head assembly for use with Union piles	—	650	650	650	650	650	650	650

TABLE IV (*continued*)

WEIGHTS OF VARIOUS PARTS OF HAMMERS AND DRIVING CAPS IN POUNDS, WITH STROKES AND ENERGIES

Size of Hammer	Normal Stroke (h)	Ram (W_r)	Casing (W_c)	Flat Anvil	Cup or Bell Anvil	Driving Head Assembly for Use with Union Piles	Anvil Block for 24" Conc. Pile	Mfr's Rated Energy in Ft. Lb. at Given Strokes Per Minute: Strokes Per Min.	Energy Per Blow (E_n)
McKiernan-Terry Double-Acting Steam Hammers									
5	7"	200	—	—	—	—	—	(300 (275 (250	(1000 (800 (650
6	8¾"	400	—	—	—	—	—	(275 (230 (200	(2500 (2160 (1680
7	9½"	800	3872	328	342	—	—	(225 (195 (170	(4150 (3720 (3280
9-B-2	16"	1500	4815	445	420	650	—	(140 (130 (120	(8200 (7000 (5940
10-B-2	20"	2500	6662	838	850	650	—	(115 (108 (100	(15000 (12500 (10700
11-B-3	20"	3625	8570	990	1060	650	—	(120 (110 (100	(22080 (18920 (15640

TABLE IV (*continued*)

WEIGHTS OF VARIOUS PARTS OF HAMMERS AND DRIVING CAPS IN POUNDS, WITH STROKES AND ENERGIES

Size of Hammer	Normal Stroke (h)	Ram (W_r)	Casing (W_c)	Flat Anvil	Cup or Bell Anvil	Driving Head Assembly for Use with Union Piles	Anvil Block for 24″ Conc. Pile	Mfr's Rated Energy in Ft. Lb. at Given Strokes Per Minute: Strokes Per Min.	Energy Per Blow (E_n)
McKiernan-Terry Double-Acting Steam Hammers									
9-B-3	17″	1600	5070	330	342	650	—	(145 (140 (135 (130	(8750 (8100 (7500 (6800
10-B-3	19″	3000	7225	625	726	650	—	(105 (100 (95 (90	(13100 (12000 (10900 (9550
11-B-3	19″	5000	8182	818	991	650	—	(95 (90 (85 (80	(19150 (18300 (17500 (16700
Special	24″	10000	18000	—	—	—	5000	90	55000
Vulcan-California Double-Acting Steam Hammers†									
E	16″	950	2850	110	118	—	—	150	3800
F	12″	330	1470	—	—	—	—	190	1300
G	8″	100	650	17	—	—	—	270	375

†Vulcan-California double-acting hammers are not now manufactured, but data included for use with existing hammers.

Note: The above weights in Table IV of caps, helmets and and anvils are typical, but may vary widely, and the actual equipment to be used should be ascertained.

After "X" has been found and put in the formula as a constant coefficient, along with the other constant coefficient "x", any value of "R" may be readily found in the field or office by using the various actual values of "s" which may occur during driving.

The simplified formula (2) for field use, in which R_u is determined after assigning constant numerical values to all terms except "s", will apply only to penetrations within a reasonable range of the value of "s" determined when establishing the formula, since with a considerably different "s" the change in value of R_u will change the values of C_1, C_2 and C_3 and thus affect the entire formula. For this reason it is often desirable to give the field office only the desired value of "s" to which the piles should be driven, and not the formula; however, if the obtained penetrations are within the same range as that specified, the simplified formula will furnish sufficiently close results. For penetrations widely different from that specified to correspond to the desired carrying capacity, the resulting R_u can be computed in the office.

Investigation of Load Capacity from Known Penetration

In the case of an investigation, where the penetration is known and it is desired to find the corresponding value of R_u, it is necessary first to assume a value of R_u for the purpose of obtaining C_2 for use in formula (1), by means of the following formula:

$$C_2 = \frac{R_u l}{AE} \tag{3}$$

and if the value of R_u then obtained by solving formula (1) varies from the assumed value a new trial should be made. It is also necessary to assume values for C_1 and C_3 from Tables I and III, and to check the resulting R_u against that used when assuming these values.

Skin Friction

A certain amount of skin friction is present on the pile during driving and the energy of the hammer blow is not all transmitted to the tip of the pile. In cases where the entire lengths of the piles are in materials of the first class, such as sands and gravels, or where the bottom or carrying stratum consists of such materials, it is immaterial whether the desired resistance is obtained by end-bearing or by skin

friction, and the value of "L" to be used in formulas (1a) or (1b) should be judged in accordance with conditions. From field observations, this value of the length seems to be quite important, and this point will be discussed later. There will not be much increase in friction over a period of time for the portions of piles driven in these materials. If any considerable portion of the pile is driven through materials of the second class, after the value of R_u is obtained, an allowance for the effects of friction may be made.[8] These effects may be additive or subtractive, in accordance with soil conditions. Where upper strata are present which are capable of picking up friction load from the pile and distributing it on material able to carry it without greater settlements than strata below, the carrying capacity of the pile increases after driving. Where a pile is driven through a hard upper layer, then through a soft layer and into a sand or gravel stratum, the friction of the upper layer might, as the soft layer settled, cause the upper layer to put additional load on the pile. For many cases, consideration will show that friction computations are unnecessary, as in the case of piles driven through deep upper strata consisting of very soft material, which neither can exert much friction during driving, nor set-up to cause much load by friction after driving, and only driven a relatively short distance into a firm stratum, so that the piles act essentially in end-bearing and may be computed by the dynamic formula above, uncomplicated by friction. It is often sufficient, particularly on small jobs where the amount of difference which might be involved in total pile footage is inconsiderable, to consider the possible effects of friction and bear these in mind when selecting the factor of safety. (This procedure was followed on many of the jobs tabulated in Appendix II).

No expression for friction values occurs in the pile driving formulas, but Hiley[9] suggests a method for evaluating friction. A rough approximation of the possible amount given by this method may be obtained by means of the following formula:[10]

$$R_f=(R_u+W_r+W_p)\times.02\times g\times L\times b \qquad (4)$$

(Note: Use the proper proportion of "L" and dimension for "g"

[8]For a tabulation of frictional values see Table VII.
[9]*Supra*, note 5.
[10]Approximated from Hiley.

TABLE V

PILE DATA

	Raymond										
	Standard Piles			Step-tapered Piles			Simplex Piles				
	22′-4″ lg.	29′-4″ lg.	37′-4″ lg.	48′-6″ lg.	56′ lg.	64′-6″ lg.	20′	30′	40′	50′	60′
Wt. of steel core, lb.	8000	9275	12000	11940-13900	13240-13900	13900	—	—	—	—	—
Tip diameter	8″	8″	8″	10″-12″	10″-11″	10″	16″	16″	16″	16″	16″
Butt diameter	17″	20″	23″	15″-17″	16″-17″	17″	16″	16″	16″	16″	16″
Uniform taper per ft.	.4″	.4″	.4″	—	—	—	0	0	0	0	0
Shell thickness	18-24 ga.	18-24 ga.	18-24 ga.	14-20 ga.	14-20 ga.	14-20 ga.	.75″	.75″	.75″	.75″	.75″
Wt. of casing, lb.	271-151	294-214	537-315	658-245	738-295	823-350	2500	3750	4950	6200	7400
Surface area, s.f. (g x L)	75	110	150	184-159	207-193	234	84	126	168	210	252

UNION MONOTUBE FLUTED PILES

	Type *F*						Type *J*				Type *Y*			
Length, ft.	20	30	40	50	60	75	10	20	30	40	10	15	20	25
Tip diameter, in.	8	8	8	8	8	8	8	8	8	8	8	8	8	8
Butt diameter, in.	10.8	12.2	13.6	14.6	16.0	18.1	10.5	13.0	15.5	18.0	12	14	16	18
Unif. taper, in. per ft.	.143	.143	.143	.143	.143	.143	.25	.25	.25	.25	.4	.4	.4	.4
Shell thickness	0 ga. (.313″), 3 ga. (.25″), 7 ga. (.188″), or 11 ga. (.125″) Note: No. 9 ga. (156″) is not standard, but can be obtained)													
Wt. of casing, lb. (based on 11 ga. Vary in prop. for other gages)	265	425	605	815	1030	1440	130	290	500	740	140	230	340	460
Surface area, s.f. (g x L)	51.5	83.0	119.0	159.5	202.5	282.5	25.0	57.5	97.5	145.0	27.3	45.3	66.5	90.3

Note: Other types of Union Monotube fluted piles available with straight shafts used on the above tapered bottom sections, from 33′ to 120′ lg. See manufacturer's catalog.

with various values of "*b*", and consider the effect of friction on only the length of pile embedded in fairly firm strata.)

In which R_f = total friction after set-up.

g = average pile girth in feet.
b = 1.0 for hard ground; 0.5 for medium ground; 0.25 for soft ground.
(For values of $g \times L$ for full lengths of Union fluted piles see Table V).

Of the total amount of friction (R_f), all of the amount occurring on soils of the first class (sands and gravels), and one-half of the amount occurring on soils of the second class (clays and fine silts) may be taken as being operative during driving and consequently included in the driving resistance. Hence, to arrive at the total bearing value of the pile, account must be taken of these amounts of friction and of the remaining friction which will occur during the set-up of soils of the second class around the pile after driving. This set-up occurs very soon following the completion of driving, and will be noticed if the piles are redriven, by observing that the penetrations are smaller than those which occurred at the stop of driving. Occasionally these penetration values are substituted in pile driving formulas for those occurring at the cessation of the continuous driving, but this should not be permitted, as the formulas were not developed nor factors of safety chosen on this basis.

After the pile lengths have been tentatively selected, but before ordering the piles, it is advisable to consider the total lengths of embedment in load-resisting friction strata and resulting carrying capacities strictly from a friction point of view, after, however, deducting such percentage for point resistance as it is felt certain will be present, as a rough check upon the designed lengths. Values of friction from actual tests are given in Table VI as an aid to judgment, lacking definite test information as to the particular case under consideration.

Factor of Safety

After the *total ultimate* carrying capacity, including such adjustments for friction as deemed proper, has been determined, this value should be divided by a suitable factor of safety to obtain the *total*

TABLE VI

TOTAL FRICTIONAL RESISTANCE OF PILES IN VARIOUS SOIL CONDITIONS

Location	Soil	Penetration (Ft.)	Contact Surface (Sq. Ft.)	Ultimate Load (Pounds)	Friction Per Sq. Ft. (Pounds)	Source of Information
			Clay and Silt			
———	Muskeg or fill	—	—	—	40	Can. Engr., Dec. 19, 1939 modified by Morrison from Kögler & Scheidig
———	Mud, wet	—	—	—	100	do
Aquia Creek, Va.[2]	Fluid mud	45-50	117	13,333	95††	Eng.-News, Feb. 23, 1893.
New York, foot of 17th St., North River[2]	Mud	50	219	33,600	130††	Proc., Int. Eng. Conf., St. Louis, 1904.
Rhine Valley[2]	Soft muck	0-35	—	—	130	Terzaghi
Rhine Valley[2]	Soft clay	25-35	—	—	295	do
Proctorsville, La.[2]	Mud, sand, clay	35	144	62,500	370††	Eng.-News, Feb. 23, 1893.
Tunis[2]	Soft muddy clay	100	—	200,000	370††	Le Genie Civil, Dec. 2, 1929.
Portland, Maine[2]	Soft blue clay	14	58.5	50,000	740††	Fay, Spofford & Thorndike
Lynn, Mass.	Harbor clay	—	—	—	750	
Boulogne-Sur-Mer	Stiff clay	—	—	—	1,800	J. S. Crandall
Hull, England[2]	Stiff blue clay	18	12"x10" timbers in sheet piling	—	1,850	Proc. Inst. of C.E., vol. LXIV, pg. 311-315
Shanghai[2]	Silty, microscopic sand	4-50	—	—	130–900	Terzaghi

TABLE VI (*continued*)

TOTAL FRICTIONAL RESISTANCE OF PILES IN VARIOUS SOIL CONDITIONS

Location	Soil	Penetration (Ft.)	Contact Surface (Sq. Ft.)	Ultimate Load (Pounds)	Friction Per Sq. Ft. (Pounds)	Source of Information
			Clay and Silt (cont.)			
Bonnet Carre, La.	Clay, sandy clay, humus and clay, sand, shells and clay, stiff blue clay	89	296.5	98,000	331	Carnegie Steel Co. "Steel Bearing Piles"
Bonnet Carre, La.	Clay, sandy clay, humus and clay, sand, shells and clay, stiff blue clay	122	490	220,000	448	do
———	Stiff clay	—	—	—	600	Can. Engr., Dec. 19, 1939, modified by Morrison from Kögler & Scheidig
———	Soft clay or silt	—	—	—	300	do
			Sand and Clay			
Monroe, La.	Sand and clay	48.5	129.5	68,000	617	Carnegie Steel Co. "Steel Bearing Piles"
Mo. Pac. Ry. Little Rock	Sand and clay	38.0	180.5	148,000	820	do
Mo. Pac. Ry. Little Rock	Sand and clay	60.0	285.0	406,940	1,430	do
Port of Bremen	Sand and clay	33.5	105.0	154,000	1,465	do

TABLE VI (*continued*)

TOTAL FRICTIONAL RESISTANCE OF PILES IN VARIOUS SOIL CONDITIONS

Location	Soil	Penetration (Ft.)	Contact Surface (Sq. Ft.)	Ultimate Load (Pounds)	Friction Per Sq. Ft. (Pounds)	Source of Information
			Sands			
Pensacola Navy Yard	White sand	16	—	—	1,900	Baker's "Masonry Construction"
Worcester, Mass.	Sand	56.5	151	130,000	860	Carnegie Steel Co. "Steel Bearing Piles"
———	Sand	—	—	—	1,000	Can. Engr., Dec. 19, 1939, modified by Morrison from Kögler & Scheidig
			Dry Gravel and Clay			
Bethlehem Steel Co.	Dry gravel and clay	36	96.1	120,000	1,250	Carnegie Steel Co. "Steel Bearing Piles"
			Sand and Gravel			
A.B. Co., Gary, Ind.	Sand and gravel	35	141	600,000	4,250	do
A.B. Co., Gary, Ind.	Sand and gravel	45	180.5	614,000	3,400	do
———	Gravel	—	—	—	1,000	Can. Engr., Dec. 19, 1939, modified by Morrison from Kögler & Scheidig

[2]From "Piles and Pile Foundations," by J. Stuart Crandall, Journal B.S.C.E., vol. XVIII, No. 5, May 1931.
††Friction calculated after deducting 15% for point resistance.

Note: For more complete data on tests from Carnegie Steel Co. see their publication "Steel Bearing Piles".

working or safe design load per pile. A factor of safety of 2 or 2½ is often satisfactory for use with this formula, and a purported factor of 6 need not be used as in the *Engineering-News* formula.

Soil Bearing

The average soil pressures at or below the level of the individual pile points, under any group of piles or under the piles in each entire footing, and under the entire structure, should in no case exceed the allowable bearing value of the soil, regardless of the values obtained for individual piles by the pile driving formula or by applying a probable square foot unit skin friction capacity to the embedded length of pile to obtain a total value. When computing the average pressure under an entire group of piles or structure, the bounding area of the group or structure often may be increased by projecting each side a distance, depending on the amount of friction embedment of the load-carrying friction length of the pile.

Analysis of Energy Losses and Check of Computations

After obtaining the value of "R_u", it is recommended that the amounts of the total applied energy lost due to various causes when driving the pile be ascertained, in order to observe the efficiency of the equipment being considered, and as a check upon the computations, by substituting the value of "R_u" in one of the following formulas:

The denominator, $s+(1/2)(C_1+C_2+C_3)$, of the first term of formulas (1a) and (1b) represents the total settlement of the pile cap under the blow of the ram. The temporary compression is covered by the term $(1/2)(C_1+C_2+C_3)$ and the permanent residual set by the term "s". Field measurements indicate that the total amount of temporary compression in the pile and ground often is not as great as computed when using the full length of the pile for "L". This is probably due to the fact that friction on the side of the pile absorbs considerable energy before it can reach the tip. Probably if reduced values of "L" and "l" were used in certain cases a closer agreement would be obtained. Rankine's formula uses ½ the pile length, but it would seem better generally to take more than this, depending on the relative amounts of friction which it is judged the various strata can exert. This depends on the material, density and depth of the

Ult. Carrying Capacity = Ultimate Resistance to Driving	Ult. Carrying Capacity Without Losses = Kinetic Energy Applied Divided by Penetration Under Last Blow	Loss in Ult. Carrying Capacity Due to Impact = Impact Loss Div. by Penetration Under Last Blow	Loss in Ult. Carrying Capacity Due to Imperfectly Elastic Compression of Pile Head & Cap = Energy Loss in Pile Hd. & Cap Div. by Penetration Under Last Blow	Pile = Energy Loss in Pile Div. by Penetration Under Last Blow	Soil = Energy Loss in Soil Div. by Penetration Under Last Blow

For Use with Drop Hammers and Single-Acting Steam Hammers

$$R_u = \frac{e_f W_r h}{s} - \frac{e_f W_r h \, W_p \frac{(1-e^2)}{W_r+W_p}}{s} - \frac{R_u C_1}{2s} - \frac{R_u^2 l}{2\,AEs} - \frac{R_u C_3}{2s} \quad (5a)$$

For Use with Double-Acting and Differential-Acting Steam Hammers

$$R_u = \frac{12 e_f E_n}{s} - \frac{12 e_f E_n W_p \frac{(1-e^2)}{W_r+W_p}}{s} - \frac{R_u C_1}{2s} - \frac{R_u^2 l}{2\,AEs} - \frac{R_u C_3}{2s} \quad (5b)$$

The above formulas are of assistance in selecting the proper weight of hammer to use, as the proportion of useful energy to wasted energy may be observed. For economical and efficient driving a reasonable proportion of the applied force should remain as the value of R_u. Furthermore, if the remaining useful force is very small, slight uncertainties in the assumptions may be as great numerically as the value of R_u, indicating that the sensitivity of the formula is too great to place too much reliance on the computed results in such cases. Both for reasons of economy and in the interests of reliance on the computed results, a hammer of efficient size should be used. It is better to select a hammer which may be on the heavy side, rather than one on the light side. In general, the hammer should be as large as can be safely used without damaging the pile, and the computed stress in the pile head should be compared with the yield point of the pile material to be sure that this value is not exceeded.

various strata, and on the relative amounts of resistance which it is judged are obtained from friction and from end bearing. For cases where the penetrations are very small and the piles are in fact practically end bearing piles, field measurements of C_2 have been observed to check the computed value of C_2 using "L" as the full length of the pile, as closely as the graph taken on the pile in the field could be measured.

Field measurements can readily be taken of the sum of C_2 and C_3, and if possible it is desirable to take these on the first piles driven, to serve as a check on the assumptions used. By holding a piece of paper on the pile just prior to the completion of driving, and moving a pencil horizontally continuously across the edge of a board, a graph will be obtained which will show the total downward movement of the top of the pile under each blow, and the amount of upward spring, represented in the formula by $(1/2)\ (C_2+C_3)$. Field measurements have been taken of C_2 alone, when driving steel II-piles to rock through material having very little friction value, and without using a driving cap, which check as closely as the graph can be made, the values of C_2 computed by the formula $C_2 = \frac{R_u l}{AE}$ and on which Table II is based. C_1 is more difficult to measure, and since a difference in this figure would make comparatively little change in the answer, it is not usually desirable to attempt to measure it. The values given in Table I are the results of field observations by Hiley[11] and Goodrich.[12] In the case of anvils having a cup to receive a driving block, common practice is sometimes to throw in short pieces of wood from time to time, which may compact to a hard layer, with the result that the value of C_1 may vary widely, and while this will not matter until the pile approaches final penetration, at that time an effort should be made to obtain consistency in the results to approximate the value of C_1 used in the formula.

Another method has been suggested by Crandall[2] for determining the proportion of the applied energy which results in permanent set of the pile, and that which is lost due to various sources. This method is applicable to field observations on driving with a drop hammer, and consists of dropping the hammer from various heights

[11] *Supra*, note 5.
[12] E. P. Goodrich—"The Supporting Power of Piles", Transactions, A. S. C. E. vol. 48, 1902.

and plotting the drop height against the penetrations obtained. By prolonging this graph, which should be a straight line, until it intersects the height axis, an intercept is obtained which gives the maximum height of fall for no penetration. This is a measure of all the energy losses for that height of fall, the only additional losses being the impact loss for any greater height of fall.

When selecting the value of C_3 for temporary compression of the soil, account should be taken of the compressibility of the strata at the pile tip and below. In the case of a soft bed under the stratum in which the tips rest, a larger value would be used than if hard material were present.

Speed of Hammer

It is important that the hammers be run at the full listed speeds, as the net available energy at the pile tip falls off rapidly at lesser speeds, particularly sharply in the case of undersized hammers. The number of strokes per minute occurring when taking final penetration readings should always be noted on pile driving reports, particularly if for any unavoidable reason the speed is less than the maximum specified, for if the speed has fallen off and is not noted, the energy is reduced, and the smaller penetrations obtained will indicate falsely high bearing values. Strokes must be counted when driving a pile, and not with the hammer running free, as the hammer speeds up at the end of driving as the penetrations become small and resistance increases.

Batter Piles

When driving batter piles the height "h" in the formula is reduced, and also friction occurs in the guides in the case of single-acting and drop hammers. Taking the coefficient of friction as 0.1, the effective value of drop "h_1" to be used in the formula may be taken as follows, where Θ = the angle between the batter and the vertical:

For drop hammers and single-acting steam hammers

$$h_1 = h\ (\cos \Theta - .1 \sin \Theta)$$

For double-acting and differential-acting steam hammers

$$h_1 = h \cos \Theta$$

Underwater Driving

When driving under water with suitable types of hammers, such as double-acting steam hammers or closed type differential-acting steam hammers, suitable compensation must be made for the buoyant effect of the water by hanging sufficient equivalent dead weight on the hammer casing; otherwise the energy of the blow will be reduced.

Uplift Piles

Piles are sometimes required to resist hydrostatic uplift on the structure, by means of friction, where the weight of the structure is insufficient to prevent flotation. For such purposes the lengths of piles should be predetermined by means of the selected safe friction values for the various strata. Pulling tests will provide this information, and the values of friction given in Table VI will give a rough indication of the range of values which may be expected. Values in Table VI are ultimate values. The factor of safety selected need not be large, and depends somewhat on the importance of the structure, frequency of the hydrostatic pressure, and expected duration of the uplift.

The total weight of earth clinging to the piles in uplift should be considered, since regardless of the magnitude of the friction value on a single pile, the piles cannot pick up more earth than is adjacent to the group and above their tips.

The design of uplift piles should provide for tension in the pile material, and suitable provision should be made for transmitting the tension into the structure.

APPENDIX I

Application of Formula to Determine Penetration and Field Formula

Sample Computations

Assumed Conditions:

Hammer—McKiernan-Terry No. 9-B-2
Blows per minute 140 (Table IV)
Energy (E_n) = 8,200 ft. lb. (Table IV)
Pile—steel pipe 10″ O.D. × ¼″ thick × 30′ lg., weight 790 lb. (to be filled with concrete after driving)
Cap—650 lb. (Table IV)
$W_p = 790 + 650 = 1440$ lb.
$W_r = 1500$ lb. (Table IV)
$e_f = .85$
$e = .4$
$R = 30$ tons (working load)
Factor of safety assumed = 2.5

Soil Conditions:
5′ top soil and recent fill
20′ soft clay
2′ sand
20′ gravel
5′ hardpan over rock

$R_u = 30 \times 2000 \times 2.5 = 150{,}000$ lb.
$C_1 = .16''$ (Table I) ($_{p1} = 150{,}000 \div 78$ sq. in. = 2000 lb. per sq. in.)
Net area of pile = 9.87″ × 3.1416 × .25″ = 7.73 sq. in.

$$C_2 = \frac{19{,}400}{15{,}000} \times .006 \times 30 = .23'' \quad \text{(Table II)} \quad (_{p2} = 150{,}000 \div 7.73 = 19{,}400 \text{ lb. per sq. in.})$$

$C_3 = .05''$ (Table III) ($_{p3} = 150{,}000 \div 78$ sq. in. = 2000 lb. per sq. in.)

$$\text{(1b)} \quad 150{,}000 = \frac{12 \times .85 \times 8200}{s + (1/2)\,(.16 + .23 + .05)} \times \frac{1500 + .4^2 \times 1440}{1500 + 1440}$$

$$150{,}000 = \frac{83{,}500}{s + .22} \times .589$$

$$s + .22 = .328$$

$$s = .328 - .22 = .108'' \text{ penetration (9 blows per inch)}$$

APPENDIX I (*continued*)

(5) Analyzing force losses:

Ult. carrying capacity without losses $\dfrac{12 \times .85 \times 8200}{.108}$	=		775,000
Impact loss $\dfrac{12 \times .85 \times 8200 \times \dfrac{1440\,(1-.4^2)}{1500+1440}}{.108}$	=	—318,000	
Cap loss $\dfrac{150{,}000 \times .16}{2 \times .108}$	=	—111,000	
Force loss in pile $\dfrac{(150{,}000)^2 \times 360}{2 \times 7.73 \times 30{,}000{,}000 \times .108}$	=	—161,000	
Soil loss $\dfrac{150{,}000 \times .05}{2 \times .108}$	=	— 35,000	—625,000
		(check)	$R_u = 150{,}000$ lb.

The short formula for field use may be obtained as follows:

$$30 \text{ tons} = \frac{X}{.11 + .22}$$

$$X = 9.9$$

(2) $$R = \frac{9.9}{s + .22} \quad \left(\text{use } R = \frac{10}{s + .22}\right)$$

in which R = safe working load in tons (due to dynamic resistance)
s = final penetration per blow in inches (use average of last 3 inches)

APPENDIX II

TABULATION OF DRIVING CONDITIONS AND FORMULAS USED FOR VARIOUS PROJECTS

Hammer			Pile					Cap										
Type	Ram Wt. lb.	Drop	Type	Lg.	Butt	Tip	Wt. Lb.	Type	Wt. Lb.	Penetration "s"	Factor of Safety	Formula	Safe Dynamic Resistance R. Tons	Total Friction Set-up, Tons	Total Permissible Design Load, Tons	Design Load Used, Tons	Eng. News Value, Tons	Strata
Drop	2280	15′	Wood	55′	14″	8″	1520	—	—	3.22″	2½	$R=\frac{51}{s+.18}$	15	0	15	15	8	33′ water, 17′ soft silt, 5′ in sand
No. 1	5000	36″	Composite	35′	15″	9.5″	1530	—	—	.59″	2½	$R=\frac{25}{s+.24}$	30	0	30	30	22	33′ water, 17′ soft silt, 5′ in sand
9-B-2	6320 ft. lb.		⅛″ Union	25.5′	13″	10″	450	2.5″ pl.	250	.025″	2½	$R=\frac{8.7}{s+.26}$	30	+12	35	30	50	16′ fill, 9½′ in sand
No. 2	3000	29″	⅛″ Union	25.5′	13″	10″	450	2.5″ pl.	250	.10″	2½	$R=\frac{12.5}{s+.26}$	34	+ 9	37	30	36	16′ fill, 9½′ in sand
No. 1	5000	36″	Raymond	31′-3″	20.5″	8″	9270	pls. and wood	150	.34″	2½	$R=\frac{13.25}{s+.09}$	30	0	30	30	34	6′ soft fill, 10.5′ in sand
No. 1	5000	36″	¼″ Union	25′	14.25″	8″	780	—	Follower 540	.66″	2½	$R=\frac{26}{s+.19}$	30	0	30	30	39	6′ soft fill, 10.5′ in sand
*9-B-2	6320 ft. lb.		Wood	30′	14″	8″	1000	—	420	.128″	2½	$R=\frac{9}{s+.25}$	24	+ 6.5	26.5		37	River mud, silt, shells, sand
*10-B-2	11460 ft. lb.		Wood	75′	14″	8″	2500	—	420	.16″	2½	$R=\frac{15}{s+.47}$	24	+ 6.5	26.5		60	30′ water, river mud, silt, shells, sand
*11-B-3	15250 ft. lb.		Wood	70′	18″	8″	2260	—	420	.655″	2½	$R=\frac{23.5}{s+.325}$	24	+ 6.5	26.5		30	30′ water, river mud, silt, shells, sand
No. 2	3000	36″	Wood	45′	12″	8″	800	—	60	.56″	2½	$R=\frac{15.3}{s+.29}$	18	0	18	18	15	35′ mud, 10′ in gravel
No. 2	3000	36″	Wood	45′	12″	8″	800	—	60	.21″	3½	$R=\frac{11}{s+.4}$	18	0	18	18	15	35′ mud, 10′ in gravel
Drop	2500	10′	Wood	45′	12″	8″	800	—	60	2.24″	2½	$R=\frac{45.5}{s+.29}$	18	0	18	18	10.5	35′ mud, 10′ in gravel
No. 0	7500	40″	10.75″ o.d. x .25″ pipe	80′	10.75″	10.75″	2270	—	675	.25″	2¾	$R=\frac{40}{s+.53}$	45	20 observed set-up in 2 weeks	45	45	143	10′ silt, 10′ sand, 40′ soft clay, 6′ in hardpan
9-B-3	7100 ft. lb.		Wood	50′	12″	8″	1100	—	—	.18″	2½	$R=\frac{10.6}{s+.35}$	20	0	20	20	25	Fill, mud, silt, sand, 9′ in gravel and sand
No. 1	5000	36″	8″ H 36 lb.	46′	8″	8″	1660	Steel, no wood	220	.66″	2½	$R=\frac{26}{s+.2}$	30			30	20	7′ clay, 25′ silt, 6′ silty sand, 8′ in sand
*No. 1	5000	36″	Raymond	31′-3″	20.5″	8″	12300	pls. and wood	225	.375″	2½	$R=\frac{13.5}{s+.15}$	26	+9.5 observed set-up in 3 days	29	35	31.5	6′ fill, 4′ sand, 13′ peat and sand, 2′ in gravel
*No. 1	5000	36″	Union	60′	16″	8″	1000	pls. and wood	225 800 follower	.375″	2½	$R=\frac{17}{s+.22}$	28.5	+6.5 observed set-up in 3 days	31	35	31.5	6′ fill, 4′ sand, 13′ peat and sand, 4′ gravel, 6′ clay, 7′ in sand

*Indicates investigation of driving results for piles driven by *Engineering-News* formula.

APPENDIX III

Summary of Comparative Results of a Test

In order to illustrate the possible effects of the proposed formula on the matter of reaching a given depth or strata and on the question of economies when determining pile lengths the following driving test results are presented:

Eleven piles were driven near each other in the same strata with a Raymond No. 1 (similar to Vulcan No. 1) single acting steam hammer having an observed 34″ stroke. Three piles were standard Raymond piles 29′-4″ long, three were No. 3 gage Union shells 25′ long with 8″ tips and 14¼″ butts, two were No. 7 gage Union shells 25′ long with 9″ tips and 15⅝″ butts, and three were Union shells 25′ long with 8″ tips and 14¼″ butts filled with Incor cement 6 days before driving. Soil conditions consisted of 4′ of cinder fill on 2′ to 4′ of marsh over fully inundated noncohesive lake sand. The ground water level was about 5′ below the surface. A pit about 2′ deep was dug for each pile to remove a frozen top layer of the fill.

A driving cap about 11½″ in diameter, consisting of a 2″ steel plate over a 6″ hardwood block over a 1″ steel plate, weighing about 150 lb., was used on each pile. In addition, a 4″ hardwood block was placed on the concrete of the prefilled piles under the lower plate. The Union piles were all driven with a follower in addition, consisting of a piece of Raymond mandrel weighing 540 lb.

The weights of piles, mandrels, caps and followers were as follows, in pounds:

	Mandrel	Pile	Cap	Follower	Total Weight
For Raymond piles	9425	250	150	—	9,825
For 8″ Union shells	—	780	150	540	1,470
For 9″ Union shells	—	600	150	540	1,290
For 8″ prefilled Union shells	—	2,560	165	540	3,265

For all piles driven to tip elevations 15′ below ground surface the following safe bearing values were computed from the observed penetrations at this depth:

APPENDIX III (*continued*)

	Penetration (Average values)	Proposed (Using F.S.=2½)	Formula Eng.-News	Eytelwein	Navy
Raymond piles	.125″	43 T	63 T	39 T	72 T
8″ Union prefilled shells (assumed $n = 10$ for concrete	.26″	40 T	39 T	33 T	46 T
8″ Union shells	.30″	43 T	35 T	38 T	44 T
9″ Union shells	.23″	46 T	43 T	50 T	58 T

In computing values by the proposed formula, values of "*L*" were taken as the distances from the heads of the piles to a point at half the distance of embedment in the sand stratum. From the above table, it appears that the proposed formula results in a maximum difference of only 13% in bearing values for the different types of piles.

In order to determine the depths to which the Union shells would have to be driven to give the same indicated bearing value as the *Engineering-News* value of 63 T for the Raymond piles, it was necessary to observe the tip elevations on the driving records at which the penetration of .125″ per blow was obtained for these piles. It was necessary to drive an 8″ Union shell 22′-6″ below ground level in order to obtain this penetration. It will be noted, however, that by the use of the proposed formula, practically identical depths of penetration would be required for any of the types of piles to obtain the same safe bearing value.

ASBESTOS-CEMENT PIPE FOR WATER MAINS AND SEWERS

By W. L. Hyland, Member*

(Presented at a joint meeting of the Sanitary Section and Hydraulic Section of the Boston Society of Civil Engineers, held on May 1, 1940)

The use of asbestos-cement pipe for water mains has been steadily increasing since it was first used for that purpose about 27 years ago, and recently the material has been made available for the construction of sewers. In view of its present widespread use it is of interest to consider some of its principal characteristics such as its physical properties, its resistance to corrosion, the methods of constructing water mains and sewers with it, its strength and watertightness, its friction coefficients and its cost as compared with the cost of water and sewer pipe of other materials.

Experience Background. Asbestos-cement pipe for water mains was manufactured first in 1913 in Italy. Its production and its use in that country increased rapidly; and statistics of the year 1931 indicate that 800 miles of it were manufactured in that year. The production of this type of water pipe in other countries was started in the year 1927; and in the United States, in the year 1930. Asbestos-cement pipe was first employed in sewer construction abroad in approximately the year 1924; and in the United States, in the year 1934. The available data shows that about 40,000 miles of this kind of water and of sewer pipes are now in service, including several thousand miles of it in the United States. Today there are two manufacturers in this country, and about twenty in other countries.

Physical Properties.[1] At present there are no universally accepted specifications for the manufacture of asbestos-cement pipe; consequently pipe made by different manufacturers differs considerably in physical properties. The pipe discussed in this paper is that of the Johns-Manville Corporation.

The pipe is made of a mixture of asbestos fibre and cement. The

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[1]Unless otherwise qualified, the statements made herein regarding the physical properties of the material in asbestos-cement pipe are based on the manufacturer's claims, and not upon independent tests made by the writer.

fibres are aligned, either circumferentially or axially but principally circumferentially, and act as an aggregate and as a reinforcement. The same kinds of materials and manufacturing processes are used in making the sewer pipe and the water pipe.

The properties of the material, of special interest to the engineer, are a bursting strength of about 5,000 pounds per square inch as determined by tests made on full lengths of pipe; a modulus of rupture in crushing of approximately 9,200 pounds per square inch as determined by the three-edge bearing method; and a modulus of rupture in flexure of about 4,600 pounds per square inch as determined by flexure tests on full lengths of pipe. The material can be stressed repeatedly up to at least 90 per cent of its ultimate strength without permanent deformation or loss of ultimate strength.

Primarily of academic interest are the axial compressive strength, about 11,900 pounds per square inch; the shearing strength, in a plane at right angles to the pipe, about 7,400 pounds per square inch; and the modulus of elasticity, about 2,400,000 pounds per square inch. The foregoing values are "average ultimate" ones, that is, the averages of a great many tests.

It should be noted here that these physical properties apply to material not bone dry nor thoroughly saturated with moisture, but of normal moisture content. Asbestos-cement is not quite as strong when wet as when dry. The loss of strength with saturation is 3 per cent of the bursting strength, 6 to 8 per cent of the crushing strength (three-edge bearings), and 14 to 15 per cent of the flexural strength.

According to the manufacturer, for design purposes the following ultimate strengths may be used:

Bursting strength—4,750 lbs. per sq. in.

Modulus of rupture in ring crushing (3-edge bearings)—7,000 lbs. per sq. in.

Modulus of rupture in flexure (pipe as a beam)—3,500 lbs. per sq. in.

In view of the importance of the bursting strength as a factor in designing the thickness of a pressure conduit, and of variations in the strength of the material as indicated by very limited data, the writer considers it advisable to use, for design purposes, a bursting

strength of 4,000 lbs. per sq. in. instead of 4,750 lbs. per sq. in. as suggested by the manufacturer.

Asbestos-cement is similar to cast iron in that neither material has a definite yield point. In comparison with the material in pit cast iron pipe asbestos-cement, as used in pipe, is about one-third as strong in bursting; about one-quarter as strong in crushing in three-edge bearings; about one-seventh as strong in flexure; and about four times as flexible as a cast iron pipe.

The material when bone dry will absorb from 14 to 18 per cent of its weight of water. This absorption is somewhat less than that of a pat of normally cured neat cement which will take up from 22 to 25 per cent of its weight of water. The material is relatively light in weight. The true specific gravities of asbestos and cement are about the same, amounting to 2.5; the pipe in which these two materials are combined has an apparent specific gravity of 1.8.

Resistance to Corrosion. A statement regarding the ability of asbestos-cement pipe to resist corrosion has been published in "Report on Transite Pressure Pipe and Couplings," by Underwriters' Laboratories, Inc., dated January 20, 1937. The Underwriters' report states in part, "The Transite Pipe and couplings dealt with in this report and installed according to the methods advocated by the manufacturer will probably withstand deteriorating influences resulting from exposure to potable waters and ordinary ground conditions for long periods of time." ("Transite" is the trade name for Johns-Manville asbestos-cement.)

The Underwriters' report describes test methods and results of chemical, of alkalinity and of leaching tests of the material in Transite pipe. These tests indicate a high percentage of silica and a comparatively low percentage of calcium. The manufacturer claims that the high percentage of insoluble silicates and the low percentage of calcium makes the pipe highly resistant to corrosion; and that, due to the higher density of the binding material, the pipe is less likely to be disintegrated by the action of acids than is a portland cement product. A further claim is that the pipe is immune to the action of soluble sulphates, and therefore is used successfully as a conduit for salt water.

As to the rubber rings that are used in jointing asbestos-cement

pipe, experience has shown that rubber used in underground joints has given long and satisfactory service, the life of rubber being favored by dampness, darkness, and absence of oxygen.

Asbestos-Cement Pipe for Water Mains

Laying and Jointing Pipe. Generally the trenches are dug sufficiently deep to afford 4′6″ to 5′0″ cover over the pipe barrel, and are about 2 feet wider than the pipe.

The pipe is furnished in 13-foot lengths in sizes 3-inches to 36-inches diameter and in 10-foot lengths in smaller sizes, and is joined together by asbestos-cement sleeve couplings and rubber rings. The pipe is light in weight, can be easily handled, and can be cut with a carpenter's saw.

Asbestos-cement pipe is laid on wooden blocks, two to a length of pipe. The blocking point is located at a distance from the end of the pipe equal to about one-fifth of its length. It is of interest to note that the manufacturer, while advocating the use of wooden blocks, also agrees that pipe laid without wooden blocks and fully supported for its entire length will carry greater trench loads. The manufacturer contends, however, that it is not practicable to shape the trench bottom to give support to long lengths of pipe for their entire length, as extreme care would have to be exercised to prevent concentrated support at the center or ends of individual pipes, with resulting stress in the pipe greater than if it were supported at the fifth points. The manufacturer also contends that the use of wood blocks permits more thorough tamping of the pipe for its full length and also facilitates rapid laying and assembly of the pipe.

The pipe couplings are easily pulled in place with the aid of an especially devised jack. When a coupling has been assembled, the rubber rings are compressed in the space between the coupling and the pipe barrel to about one-half the original thickness. Friction prevents them from being moved by the water pressure. A typical pipe coupling is shown in Fig. 1.

The pipe fittings used in asbestos-cement pipe lines are cast iron with bell ends. The joints in the cast iron bells are made with sulphur base compound or lead, just as they are in a cast iron pipe line. Oversize cast iron bells are required for pipe larger than 8 inches diameter, but standard size bells can be used for pipe 8 inches and smaller.

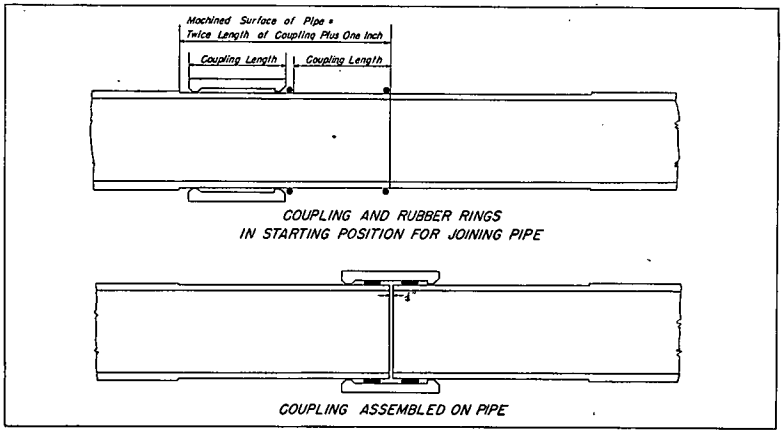

FIG. 1.—TYPICAL COUPLING FOR ASBESTOS-CEMENT PRESSURE PIPE

Half-lengths of asbestos-cement pipe are used in each bell of a cast iron fitting or in a valve, to minimize flexural stresses. Each pipe fitting is backed up with a block of concrete bearing against the undisturbed side of the trench, to distribute the thrust due to hydrostatic pressure.

The pipe lines are backfilled by hand to a height of at least 2 feet above the pipe barrel, except at the joints, immediately after the pipe is laid but before water pressure is applied. Care must be taken not to allow stones to fall on the pipe as they are apt to cause an indentation in the pipe at the point of impact. The backfill is rammed around the pipe with railroad tamping irons, and all of the pipe fittings are properly braced. The individual pipes and the fittings are thus restrained against movement caused by water pressure.

Dead end mains from which water is drawn at low rates and intermittently, sometimes show a tendency to impart harshness to the water, when the mains are not flushed or bled. In anticipation of this condition, dead ends are sometimes given an interior bituminous coating.

Connections for service pipes are made by tapping the main in exactly the same way as is done with cast iron pipe. It is Fay, Spofford & Thorndike's custom to tap a pipe at about its horizontal

diameter, and to provide a gooseneck in the horizontal plane in the service pipe to take up any settlement or movement of either pipe.

Strength. In discussing the strength and also the watertightness of asbestos-cement water mains, the writer has drawn on his experience in connection with water systems designed by and constructed under the supervision of Fay, Spofford & Thorndike, and in which asbestos-cement pipe was used exclusively. One of these was at Sterling, Massachusetts, where 3 1/3 miles of pipe were installed. Another at Narragansett, Rhode Island, included 9 miles of Transite pipe. A third, an extension to the Wareham, Massachusetts, water system, included 3 miles of the pipe.

The pipe for the Sterling job was classified as Class C, which was rated at a 130 lbs. per sq. in. working pressure. Each length of pipe was tested at the factory at 260 lbs. per sq. in. hydrostatic pressure. In the field, after backfilling, the mains were tested at a pressure within a range of 150 to 265 lbs. per sq. in., depending upon the elevation of the water mains. During the field tests, 33 pieces of pipe or 1¾ per cent failed out of 1,839 pieces of pipe tested; two couplings failed out of 1,607 couplings tested; and three rubber rings blew out, out of 3,214 rubber rings installed. Two couplings leaked during the tests; one due to a misplaced rubber ring, and the other due to a scar on the pipe under a rubber ring. (Other field tests which were made to measure leakage at a lower pressure will be referred to later.)

Subsequent to the time that the Sterling job was constructed, hydrostatic pressures used in factory tests of the pipe were increased from two to four times the maximum rated working pressure. In addition, each pipe of 8-inch size and smaller is now subjected to a flexural test.

Pipe for the Narragansett job was Class 150, which is suitable for 150 lbs. per sq. in. working pressure. At the factory, each length of pipe was tested at 600 lbs. per sq. in. hydrostatic pressure, and each 6-inch and 8-inch pipe was tested in flexure under a load sufficient to develop a fibre stress of 3,000 lbs. per sq. in.

The mains were subjected to a hydrostatic test of 200 lbs. per sq. in. in the field after laying. In the field tests, six pipes or about ⅛ of one per cent failed out of 3,860 pieces of pipe tested; two coup-

lings failed out of 3,652 couplings tested and five rubber rings blew out, out of 7,304 rubber rings installed. Three couplings leaked during the tests; one due to a scar on the pipe under a ring, another due to a misplaced rubber ring, and the third due to a small root under a rubber ring. (Other field tests which were made at a lower pressure to measure leakage will be referred to later.)

During the aforementioned tests, no single length of pipe showed any signs of sweating under pressure.

The Water Commissioners at Sterling report that since 1935, when the Sterling system was constructed, there have been no breaks or visible leaks in the mains. The Water Commissioners at Narragansett report that since June, 1939, when the Narragansett water system was completed, there have been no breaks or visible leaks in the water mains.

Water-tightness. Tests for leakage were made at Sterling under 80 lbs. per sq. in. pressure; and at Narragansett, under 100 lbs. per sq. in. pressure, after the trenches had been completely backfilled.

The average leakage for the entire Sterling system, which was measured in a single test, was 35 gallons per day per mile of pipe per inch of diameter, or 256 gallons per day per mile.

The weighted average rate of leakage for the entire Narragansett system, combining the results obtained by testing the system in sections, was 46 gallons per day per mile per inch of diameter, or 406 gallons per day per mile. The minimum rate of leakage was found to be 6 gallons per day per mile per inch of diameter, or 60 gallons per day per mile; and the maximum rate of leakage was 89 gallons per day per mile per inch of diameter, or 890 gallons per day per mile. The results of the leakage tests are given in Table 1.

It is interesting here to note a comparison between these results and actual results obtained from like tests on cast iron pipe lines, drawn from the writer's experience. The latter tests were made on about 160 miles of cast iron pipe lines having bell and spigot joints sealed with sulphur base compound. The joints had been subjected to water pressure continuously for 30 days or longer before the tests were made, and they had undergone the healing or tightening process characteristic of compound joints. The average rate of leakage from the cast iron mains was 0.5 gallons per day per linear foot of joint.

TABLE 1

RESULTS OF LEAKAGE TESTS

Pipe Tested Length Feet	Pipe Tested Size In.	Number of A. C. Sleeve Couplings 6-in.	8-in.	10-in.	Number of Poured Compound Joints 6-in.	8-in.	10-in.	Pressure Used in Testing Lbs. per Sq. In.	Leakage GPD per ml. per 1-In. Diam.	Leakage GPD per Mile
				Sterling, Mass.						
9486	6	944	—	—	156	—	—			
4683	8	—	380	—	—	61	—			
3586	10	—	—	283	—	—	24	80		
		3.36 Miles of Pipe							35	256
				Narragansett, Rhode Island						
2558	6	200	—	—	75	—	—			
3856	8	—	300	—	—	32	—			
1256	10	—	—	97	—	—	17	100		
		1.45 Miles of Pipe							82	625
4359	10	—	—	340	32	—	28	100		
		0.83 Miles of Pipe							89	890
1807	6	140	—	—	77	—	—			
1446	8	—	110	—	—	11	—			
6858	10	—	—	540	—	—	28	100		
		1.91 Miles of Pipe							51	460
1700	6	140	—	—	66	—	—			
1675	8	—	125	—	—	7	—			
3486	10	—	—	270	—	—	24	100		
		1.30 Miles of Pipe							20	170
1487	6	120	—	—	35	—	—			
2881	10	—	—	230	—	—	18	100		
		0.83 Miles of Pipe							14	122
1329	6	110	—	—	41	—	—			
2857	8	—	230	—	—	30	—			
118	10	—	—	10	—	—	7	100		
		0.814 Miles of Pipe							45	330
6108	10	—	—	480	40	2	30	100		
		1.16 Miles of Pipe							6	60
2660	10	—	—	210	23	—	33	100		
		0.5 Miles of Pipe							78	780
		8.8 Miles of Pipe (Complete System)						100	46	406

Had the Sterling and Narragansett systems been constructed of bell and spigot pipe in 16 foot lengths with sulphur base joint compound, and had the leakage been at the aforementioned rate of 0.5 gallons per day per linear foot of joint, the rate of leakage from the Sterling mains would have been about 400 gallons per day per mile instead of 256, and from the Narragansett mains about 500 gallons per day per mile of pipe instead of 406.

Value of Hazen-Williams Coefficient "C". For asbestos-cement pipe the value of "C" as used in the Hazen-Williams formula is relatively high. At Sterling it was found by tests to be about 145 for pipe lines containing several bends and special cast iron fittings, and about 150 for straight lines containing few special cast iron fittings. At Wareham, Mass., it was found to be about 148 for an 8-inch pipe line containing few special cast iron fittings.

The interior surface of asbestos-cement water pipe does not become roughened by tuberculation and the original high value of "C" appears not to diminish with age.

Comparative Costs of Pipe and Installation. The cost of asbestos-cement pipe installed generally has been somewhat less than that of cast iron pipe in a comparable class. At Sterling and Narragansett, comparative bids were received for asbestos-cement and cast iron water mains. At Sterling, the bids were based on what was then termed Class C Transite Pipe, which was classified as being suitable for 130 pounds working pressure; and for Class 150 cast iron pipe to be furnished under Federal Specifications WW-P-421. There was to be no special lining inside of the cast iron pipe. At Narragansett, the bids were based on Class 150 Transite Pipe and Class 150 Cast Iron Pipe, the latter to be furnished under Federal Specifications WW-P-421. Cast iron pipe was to have an inside cement mortar lining. Comparative costs based on the successful bids for the Sterling and Narragansett projects are given in Table 2.

Factor of Safety. It is of interest to note the factor of safety afforded by asbestos-cement pipe as compared with that of the most widely used material for water mains, cast iron pipe. Prior to the publication by the American Standards Association of "Manual for the Computation of Strength and Thickness of Cast Iron Pipe" and "Specifications for Cast Iron Pit Cast Pipe for Water or Other

TABLE 2

COMPARATIVE BIDS FOR EXCAVATING AND BACKFILLING TRENCH, FURNISHING, LAYING AND TESTING PIPE

(Dollars per Linear Foot of Water Main)

Size of Pipe	Sterling, Mass.		Narragansett, R. I.	
	Asbestos-Cement Pipe	Cast Iron Pipe	Asbestos-Cement Pipe	Cast Iron Pipe
10 inch	$2.00	$2.30	$1.80	$1.95
8 inch	1.65	1.80	1.42	1.50
6 inch	1.27	1.40	1.10	1.12

COMPARATIVE BIDS FOR COMPLETE DISTRIBUTION SYSTEM

(Excluding Source of Supply and Storage Tank)

Sterling, Mass.		Narragansett, R. I.	
21,600 ft. of 10", 8" and 6" Pipe		44,000 ft. of 10", 8" and 6" Pipe	
Asbestos-Cement Pipe	Cast Iron Pipe	Asbestos-Cement Pipe	Cast Iron Pipe
$46,689 or $2.16 per ft.	$50,194 or $2.32 per ft.	$95,840 or $2.18 per ft.	$101,082 or $2.30 per ft.

Liquids," in December, 1939, there was no commonly accepted method for determining the ability of a pipe to withstand the combined stresses due to hydrostatic pressure, water hammer, earth loads, and live loads. Sectional Committee, A21, of the American Standards Association, which has been engaged since 1926, in the preparation of the Manual and of the new Specifications, has produced a new theory of design in which not only static water pressure and water hammer, but also earth loads and load from trucks are included in computations for pipe thickness. Different methods of supporting the pipe also are taken into consideration. The methods outlined in this Manual have been utilized herein in computing the Factors of Safety afforded by asbestos-cement pipe as compared with those for cast iron pipe.

Factors of safety for pipe in service have been computed for Class 150 asbestos-cement pipe as made by Johns-Manville Corporation, and for Class 150 pit cast pipe, made under American Standards Association Specifications for Cast Iron Pit Cast Pipes for Water or Other Liquids. Pit cast iron pipe has been used for the comparison rather than centrifugal cast iron pipe because uniform design stresses

for pit cast pipe have been agreed upon and are published in the aforementioned "Manual"; whereas there are as yet no uniformly accepted design stresses for centrifugal cast iron pipe. The American Standards Association is now preparing a new basis for design and new specifications for centrifugally cast iron pipe, and it is expected that they will be published in the near future.

TABLE 3

FACTORS OF SAFETY FOR PRESSURE PIPE

	Asbestos-Cement Pipe Class 150			Pit Cast Iron Pipe Class 150 A.S.A. Specs. A21.2-1939		
	Minimum	Factor of Safety			Factor of Safety	
Size of Pipe Inches	Wall Thickness at Machined Ends of Pipe-Inches *	Trench Condition B (Pipe on Flat Bottom Trench-Tamped)	Trench Condition D (Pipe on Blocks-Tamped)	Minimum Wall Thickness Inches **	Trench Condition B (Pipe on Flat Bottom Trench-Tamped)	Trench Condition D (Pipe on Blocks-Tamped)
6	.54	2.41	2.17	.28	3.32	2.96
8	.61	2.01	1.80	.31	2.74	2.39
10	.78	2.09	1.89	.38	2.69	2.34
12	.90	2.08	1.88	.42	2.54	2.17
14	1.03	2.05	1.85	.52	2.77	2.43
16	1.15	2.06	1.88	.57	2.72	2.36
20	1.43	2.02	1.89	.67	2.59	2.29
24	1.77	2.15	1.94	.77	2.61	2.36

*Minimum wall thicknesses are obtained by deducting from nominal wall thicknesses of machined ends of pipe .08 inch manufacturing tolerance for 6 inch to 12 inch pipe; and .10 inch for larger pipe.

**Minimum wall thicknesses are obtained by deducting from nominal wall thicknesses a corrosion allowance of .08 inch, and foundry tolerances of .07 inch for 6 inch and 8 inch pipe; and .08 inch for 10 inch to 24 inch pipe.

In computing the factors of safety, which are given in detail in Table 3, the following assumptions have been made:

The width of trench is two feet greater than the inside diameter of the pipe. The depth of earth cover over the pipe is five feet. Two field conditions are assumed: pipe laid on flat bottom trench and tamped (field condition B in the "Manual"), and pipe laid on blocks and tamped (field condition D). The rated static pressure for each

kind of pipe is 150 lbs. per sq. in. The assumed magnitudes of water hammer are those which have been used in the A. S. A. Specifications, which are as follows:

Diameter of Pipe	Water Hammer Lbs. per Sq. In.
6 inches to 10 inches	120
12 inches and 14 inches	110
16 inches	100
20 inches	90
24 inches	85

The design strength in bursting and the design moduli of rupture in crushing and in flexure are as follows:

	Asbestos-cement	Pit Cast Iron
Bursting Strength	4,000 lbs. per sq. in.	11,000 lbs. per sq. in.
Crushing Strength (Modulus of Rupture in crushing as a ring, in three-edge bearings)	7,000 lbs. per sq. in.	31,000 lbs. per sq. in.
Flexural Strength (Modulus of Rupture—pipe as a beam)	3,500 lbs. per sq. in.	26,000 lbs. per sq. in.

The design strengths used for pit cast iron pipe in the above tabulation are conservative values; they are about 20 per cent less than average values obtained in tests, and are given as design strengths in the new A. S. A. Manual. The strengths used for asbestos-cement pipe are discussed under "Physical Properties" in the preceding text.

The weight of earth is considered to be 120 pounds per cubic foot. The live load is considered to be two passing trucks with rear axles both over the pipe at the same time, adjacent wheels of the two trucks three feet apart center to center, and load on each rear wheel 9,000 pounds. To the load thus obtained, 50 per cent is added for impact of the moving trucks over a road or a trench surface assumed to be rough. In computing the factors of safety, the thickness at the machined end of the pipe has been used for asbestos-cement pipe,

and the manufacturing tolerance has been deducted from it; the foundry thickness tolerances and the corrosion allowances given in the A. S. A. Manual, have been deducted from the thickness of cast iron pipe.

It is assumed that the pipe should be able to withstand successfully the stresses due to rated static pressure plus fill load, plus water hammer; or rated static pressure plus fill load, plus live load. It is assumed that the pipe will not be subjected to live load and to water hammer simultaneously.

Based on the above conditions, the factors of safety afforded by asbestos-cement pipe and cast iron pipe are given in Table 3.

The system of loading that was found to produce the least factor of safety was static pressure plus fill load plus water hammer. The factors of safety given in Table 3 are predicated upon this system of loading, and upon stresses resulting from ring crushing and bursting. Bending is not a factor except in 6-inch asbestos-cement pipe figured as a beam spanning between blocks, for which condition the factor of safety is 2.

Asbestos-Cement Pipe for Sewers

Within the past few years asbestos-cement pipe has been available also for the construction of gravity sewers. The sewer pipe is manufactured in the same manner as the water pipe; and, like the water pipe, is furnished in 13 foot lengths. The ends of the sewer pipes are not machined, as are the ends of the water pipe. Instead of rubber rings, bituminous material furnished by the manufacturer of the pipe is used in the joints.

For sewage force mains, it is customary to use the same pressure pipe that is used for water mains.

The background for this part of the paper, which deals with sewer pipe, was obtained primarily in connection with the new sewer system for Cranston, Rhode Island, which was designed and is being constructed under the supervision of Fay, Spofford & Thorndike. For this project, there has been purchased to date about two miles of 14-inch and 20-inch asbestos-cement sewer pipe, and about one mile of 10-inch and 16-inch asbestos-cement pressure pipe for force mains, out of a total of about fifty miles of pipe purchased. About one-half of the asbestos-cement pipe purchased has been laid.

Laying and Jointing. Asbestos-cement sewer pipe is obtainable in four thickness classes, 1 to 4; and in sizes 4-inches to 36-inches diameter. Class 1 is the lightest weight, and Class 4 is the heaviest.

The sewer pipe is laid on wooden blocks in much the same manner as the water pipe. It appears to be advisable not to place the blocks on unyielding material, but to bring them to the proper grade by filling and tamping the soil under them. There should be a slight yielding of the blocks as the load comes on to the pipe, permitting the pipe definitely to settle to the granular material in which it is embedded. Half lengths of pipe are used to connect to a manhole, to minimize flexural stresses.

A concrete cradle for the pipe is placed wherever trench loads warrant the use of it. Where a concrete cradle is not used, the pipe is embedded in granular material. When gravel stone is used under a pipe for underdrain or for any other purpose, the lower quadrant of the pipe is embedded in it and a finer granular material is used above the level of the stone. The following three types of bedding for pipe in the trench are provided for by the Cranston specifications:

1. A well compacted bedding of granular material
2. A concrete cradle encasing the lower part of the pipe, called "part concrete cradle"
3. A concrete cradle completely encasing the pipe, called "full concrete cradle"

These types of bedding are illustrated in Figure 2.

Pipe bedded according to the first method, in well compacted gravel, can support from 1½ to 2 times the load that it can carry in 3-edge bearings. Pipe bedded in part concrete cradle can support about 2½ times the 3-edge bearing loads, and pipe completely encased in concrete can be subjected to a still greater load which may be carried principally by the concrete casing. These ratios of the strength of a pipe in a trench to its strength in 3-edge bearings are called "load factors".

In making a joint, the ends of the pipe are separated about ⅛ inch to allow for expansion; and a band of special tape is applied to seal the small space between them. The sleeve coupling is slid into place and blocked up in such a manner as to provide a uniform annular space for the joint compound. The spaces at both ends of the

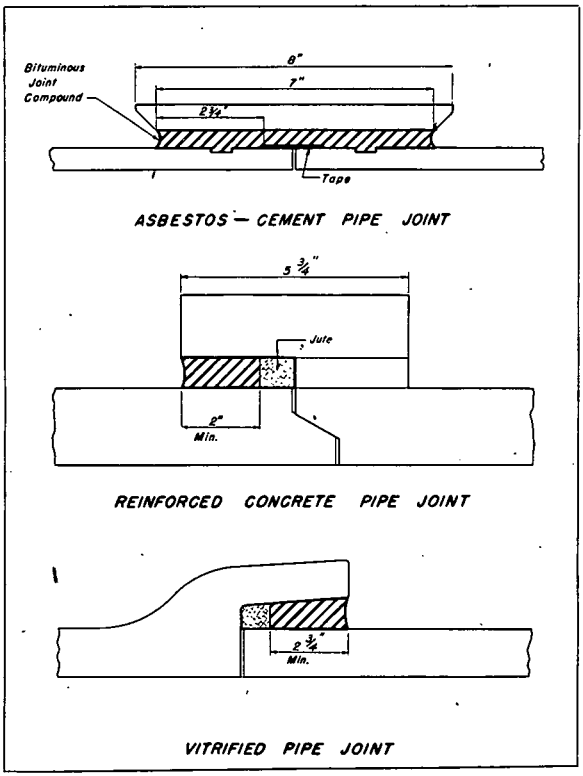

FIG. 2.—TYPICAL JOINTS FOR SEWER PIPE

coupling are closed with so-called snakes; a pouring cone is placed at the top of each snake; and the bituminous material is poured into one of the cones until it has filled the joint and has risen to the tops of both cones. A good bond is obtained between the bituminous joint compound and the pipe by means of a priming coating which is applied at the factory to the ends of the pipe and to the inside of the couplings. Typical sewer pipe joints are illustrated in Figure 3.

Service Connections. The manufacturer recommends that connections to asbestos-cement sewer pipe be made by means of a cast iron Y or T fitting or "connector". A hole is cut in the desired location in the pipe with a chisel and a hammer; the cast iron connector is inserted into the hole; and the pipe and the connector are wrapped

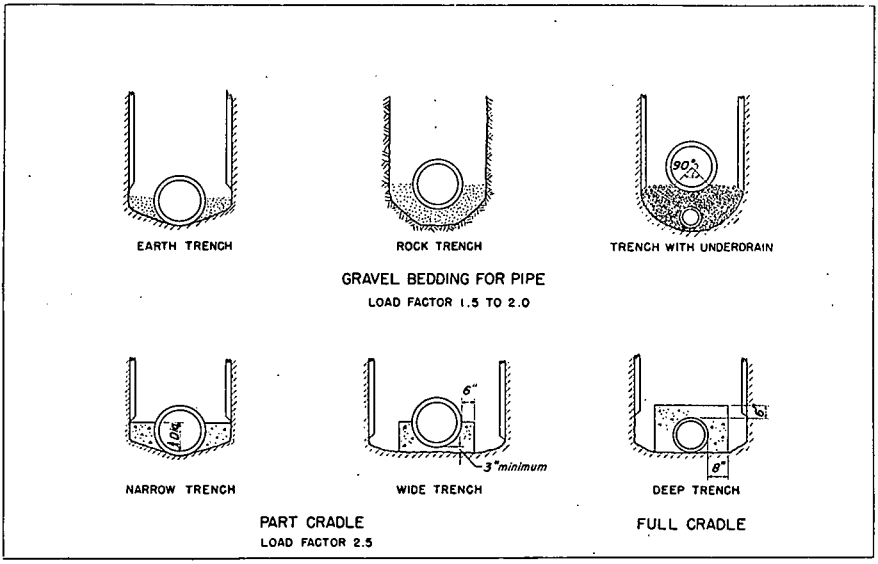

FIG. 3.—TYPICAL SECTIONS OF TRENCH

with steel mesh reinforcing and cemented in place. On the Cranston job, however, it was found that these connectors frequently were damaged by rough treatment; and a type of connection was devised whereby the cast iron Y or T fitting, instead of being held in place merely by a reinforced cement jacket, is bolted to the pipe. The cast iron fitting has a flange similar to the rim of a hat which fits the shape of the pipe. A hole is cut in the pipe to fit the opening in the fitting; bolt holes are drilled through the pipe to match the bolt holes on the flange; and the fitting is bolted to the pipe, using monel metal bolts. A heavy rubber gasket is used between the flange and the pipe.

Water-Tightness. There is very little information available as to the rate of infiltration of ground water into asbestos-cement pipe sewers. At Cranston the rate of infiltration was measured for a section of 14-inch sewer including about 3200 feet of 14-inch asbestos-cement pipe and 410 feet of 15-inch reinforced concrete pipe, all of which was laid at a depth of 2 or 3 feet below ground water level. This sewer was constructed by WPA forces and was the first section of asbestos-cement pipe sewer constructed on the project. The

measured rate of infiltration was 15,700 gallons per mile per day. The Cranston specifications require that, regardless of size or kind of sewer, the infiltration shall not exceed 30,000 gallons per mile per day.

In making comparative estimates of the amount of infiltration into other sewers as compared with asbestos-cement pipe sewers, the depth of joint and the number of linear feet of joint per unit length of sewer are the principal factors to be considered. Combining the effects of these two factors, the infiltration for different kinds of sewers may be compared by means of the following ratios, the lowest ratio indicating the lowest rate of infiltration:

		Ratio
Asbestos-Cement Pipe Sewer—	13 ft. Pipe Lengths	1
Reinforced Concrete Pipe Sewer—	8 ft. Pipe Lengths	1¼
Reinforced Concrete Pipe Sewer—	4 ft. Pipe Lengths	2½
Vitrified Pipe Sewer—	3 ft. Lengths	3⅛

Value of Kutter's Coefficient "n" for Sewer Pipe. The inside surface of asbestos-cement sewer pipe is equally as smooth as that of asbestos-cement water pipe and the same friction coefficients should apply for both provided the conduit is clean and unobstructed. Hazen & William's coefficient "C" has been found to be 145 to 150 for the water pipe and the equivalent value of Kutter's coefficient "n" is about .010. There is practically no data available, however, as to measured values of "n" for existing asbestos-cement gravity sewers. For the Cranston job, Kutter's "n" of .013 was used for asbestos-cement pipe, this value being the one commonly used for reinforced concrete pipe sewers.

Comparative Cost of Pipe and Installation. Comparative costs based on average bids received during the past year for asbestos-cement pipe, reinforced concrete and vitrified pipe for the Cranston Sewer System are given in Table 4. The classes of pipe for which the costs are given are as follows: asbestos-cement pipe, Class 1 in 6-inch, 8-inch and 10-inch sizes, and Class 2 in larger sizes; reinforced concrete pipe, A. S. T. M. Designation C76-37 for so-called "culvert" pipe; vitrified pipe, A. S. T. M. Designation C13-35 with certain modifications.

The cost data includes cost of pipe, joint materials and Y-

TABLE 4

COMPARATIVE COSTS OF SEWER PIPE

(Including Joint Material and Branches)

Size of Pipe	Asbestos-Cement	Reinforced Concrete Culvert Pipe	Vitrified Clay Pipe
6 inch	$ .36 (Class 1*)	—	$ .18
8 "	.70 "	—	.28
10 "	.79 "	—	.43
12 "	1.03 (Class 2*)	—	.60
14 "	1.43 "	—	—
15 "	—	1.17	1.51
18 "	1.96 "	1.54	1.81
20 "	2.38 "	1.88	2.26
24 "	3.16 "	2.33	2.56
30 "	4.14 "	3.38	—

*Johns-Manville Classification

branches. The cost for vitrified clay pipe also includes an allowance of $6.00 per cubic yard of concrete for concrete cradle to be placed only where required to bolster the strength of the pipe.

No data is available, from the Cranston project, regarding comparative costs of installation of the various kinds of pipe. It is unlikely, however, that the installation costs for the three kinds of pipe differ appreciably.

The greater part of the pipe purchased for the Cranston sewer system was awarded without including in the comparison of bids an evaluation of the cost of pumping and of treating infiltration. An evaluation of infiltration generally has been included, however, in comparing bids for pipes larger than 12-inch diameter, used in places where it was known that they would be laid below ground water level. Infiltration was not considered in evaluating bids for pipe 12 inches and smaller in diameter, because these sizes generally are used in the upper reaches of the sewer system, are laid at shallower depths than larger sizes of pipes, and most frequently are laid in ground that is dry or usually dry.

Strength of Asbestos-Cement Sewer Pipe. Each length of the asbestos-cement sewer pipe for the Cranston job was tested at the factory at hydrostatic pressures of 200 lbs. per square inch for Class 1

pipe and 300 lbs. per square inch for Class 2 pipe. Each length of 6-inch and 8-inch pipe was tested also in flexure at the factory, under loads sufficient to develop a fibre stress of at least 3000 lbs. per square inch. Ultimate strengths for Class 1 and Class 2 asbestos-cement sewer pipe in 3-edge bearings are given in Table 5. According to the manufacturer of the pipe these strengths are average values of a great many tests.

TABLE 5

ULTIMATE STRENGTH OF ASBESTOS-CEMENT SEWER PIPE
(A.S.T.M. 3-EDGE BEARING TESTS)

Size of Pipe Inches	Class 1	Class 2
	Lbs. per Linear Foot	
6	2880	—
8	3100	—
10	2580	3690
12	2370	3850
14	2200	3920
16	2120	4050
18	2030	4140
20	2290	4280
24	2340	4550
30	2980	5000
36	3500	5400

It is interesting to note that the strength of Class 1 pipe is about 30 per cent less and of Class 2 pipe is 25 per cent more than that of A. S. T. M. C75-35 Reinforced Concrete Sewer Pipe; and that Class 1 pipe is about 50 per cent as strong as, and Class 2 pipe is practically as strong as, A. S. T. M. C76-37 Reinforced Concrete Culvert Pipe.

Class of pipe and kind of bedding are selected with a view to providing a pipe line which will be strong enough to afford a surplus strength of about 50 per cent, or a factor of safety of 1½.

The safe load that a pipe in a trench can carry is the product of the ultimate strength of the pipe in 3-edge bearings and the load factor,* divided by the factor of safety. Safe trench loads, based on a factor of safety of 1.5, for Class 1 and Class 2 asbestos-cement sewer

*Load factor is the ratio of the strength of a pipe in a trench to its strength when tested by the 3-edge bearing method.

pipe with gravel bedding, and also with part concrete cradle bedding, are given in Table 6.

TABLE 6

SAFE LOADS FOR ASBESTOS-CEMENT SEWER PIPE IN TRENCH

(Pounds per Linear Foot of Pipe)

Size of Pipe	Class 1		Class 2	
	Load Factor 1¾ Gravel Bedding	Load Factor 2½ Part Concrete Cradle Bedding	Load Factor 1¾ Gravel Bedding	Load Factor 2½ Part Concrete Cradle Bedding
6	3360	4800	—	—
8	3620	5170	—	—
10	3020	4300	4300	6150
12	2770	3950	4500	6410
14	2570	3670	4580	6540
16	2470	3530	4660	6660
18	2370	3390	4830	6900
20	2670	3800	5000	7140
24	2730	3900	5310	7590
30	3480	4970	5840	8340
36	4090	5840	6300	9000

Earth loads on pipe in a trench have been computed, based on the widely used theories developed during the past 30 years by Iowa Engineering Experiment Station, and are presented here in a form which has been found to be convenient for practical application of the data. In applying these theories, the weight of fill has been assumed to be 120 pounds per cubic foot; values of friction and other coefficients that have been used are those that give probable maximum loads and are those that are frequently used when the character of the soil is not definitely known. Trench loads for various sizes of pipe, various trench widths and various depths of cover are shown in Fig. 4.

The diagram in Fig. 4 may be used as follows: To determine the earth load on a given size of pipe for a given width of trench and a given depth of cover, enter the diagram at the given trench width at the bottom of the diagram and go vertically to the curve for the given depth of cover. If the point so found is to the left of the

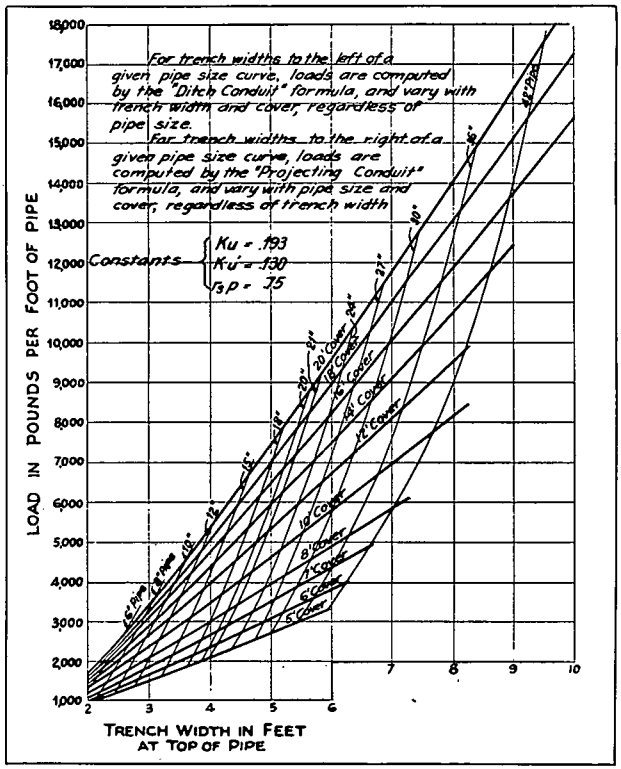

FIG. 4.—EARTH LOADS ON SEWER PIPE IN TRENCHES

nearly vertical curve for the given pipe size, the "ditch conduit" theory applies and the load on the pipe is read or interpolated directly opposite the point. If the intersection of the trench width with the depth of cover curve is to the right of the curve for the particular size of pipe, the "projecting conduit" theory applies and the depth of cover curve should be followed downward to the left to the intersection with the curve for the given pipe size and the load can then be read or interpolated directly opposite this point.

The live load on a pipe in a trench has been computed assuming two passing trucks with rear axles both over the pipe at the same time, adjacent wheels of the two trucks 3 feet apart, center to center, and load on each rear wheel 9000 pounds; 50 per cent has been

added for impact assuming the road surface to be rough. Experiments made at the Iowa Engineering Experiment Station have shown that loads on pipe, due to a concentrated superload, will be practically the same whether the pipe is a ditch conduit or a projecting conduit and regardless of the width of trench. The live load

TABLE 7

LIVE LOAD ON PIPE

(Pounds per Linear Foot of Pipe)

Depth of Cover Over Pipe	Size of Pipe 6″	8″	10″	12″	14″	18″	20″	24″	30″	36″
6 feet	140	240	300	350	410	530	590	710	870	1020
8 "	100	160	190	240	280	370	410	490	600	700
10 "	70	110	140	170	200	260	290	350	430	510
12 "	50	80	110	120	140	190	210	260	330	390
14 "	—	60	80	100	120	160	170	200	260	310
16 "	—	50	70	80	100	130	140	170	220	260
18 "	—	—	60	70	80	110	120	140	180	220
20 "	—	—	—	60	70	100	110	120	160	190
22 "	—	—	—	—	60	90	100	110	140	170

TABLE 8

MAXIMUM PERMISSIBLE DEPTHS OF COVER OVER ASBESTOS-CEMENT SEWER PIPE

(Factor of Safety = 1.5)

Size of Pipe	Widths of Trench*	Class 1 Pipe: Gravel Bedding Load Factor 1.75	Class 1 Pipe: Part Concrete Cradle Bedding Load Factor 2.5	Class 2 Pipe: Gravel Bedding Load Factor 1.75	Class 2 Pipe: Part Concrete Cradle Bedding Load Factor 2.5
8 inch	3′-8″	20 feet	24 feet		
10 "	3′-8″	14 "	19 "	19 feet	28 feet
12 "	3′-8″	10 "	15 "	18 "	28 "
14 "	4′-0″	8 "	12 "	14 "	25 "
18 "	4′-0″	5 "	8 "	16 "	27 "
20 "	4′-2″	5 "	9 "	15 "	27 "
24 "	5′-0″	4 "	7 "	11 "	20 "
30 "	6′-0″	5 "	8 "	9 "	16 "
36 "	7′-0″	4 "	8 "	9 "	13 "

*Widths of trench used on the Cranston Sewer Project.

transmitted to one linear foot of pipe in a trench is given in Table 7.

Using the safe loads given in Table 6, and the earth and live loads given in Fig. 4 and Table 7, the permissible depths of cover for asbestos-cement sewer pipe can be obtained and they are given in Table 8. The widths of trench given in this tabulation are the ones used on the Cranston Sewer Project.

Conclusions. Because asbestos-cement pipe has had a satisfactory, even though relatively short experience as a material for water mains and sewers, and in view of its several attractive qualities considered in this paper, it should be given serious consideration in the selection of pipe materials for water mains and sewers.

Acknowledgment. The writer wishes to acknowledge the assistance received from his associates in the firm of Fay, Spofford & Thorndike, and from representatives of pipe manufacturers, in preparing this paper.

THE FILTRATION SYSTEM FOR THE NEW M.I.T. SWIMMING POOL—DESIGN AND OPERATION

THOMAS R. CAMP, Member*

(Presented at a meeting of the Sanitary Section of the Boston Society of Civil Engineers held on December 4, 1940)

ALUMNI POOL (Fig. 1) is a product of "Tech." Its construction was made possible by contributions from the alumni, and its design was perfected by the joint efforts of some 50 members of the faculty under the able leadership of Professors Anderson and Beckwith, the architects. All the resources of the technical staff of the Massachusetts Institute of Technology have been pooled in an effort to make Alumni Pool one of the finest swimming pools in the country. The pool is being operated by the staff of the Superintendent of Buildings and Power under the general direction of a Sanitation Committee composed of members of the faculty.

Alumni Pool is intended for the general use of students and staff, but its design is such as to facilitate the holding of swimming meets. Two pools have been provided in the building (Fig. 2), a large pool for general swimming and diving and a small practice or beginners' pool. A spectators' gallery has been built along the north side of the large pool to accommodate about 340 persons. Dressing and locker facilities for men have been provided in the corridor under the spectators' gallery, and a temporary enclosed corridor has been constructed to connect the pool building with Barbour Field House. Most of the men bathers will use the lockers in Barbour Field House. Women bathers have a dressing room and showers on the second floor of the pool building above the coaches' office. Men and women bathers have separate entrances from the showers to the pool enclosure, and each entrance corridor is provided with a foot bath.

There are many interesting features in the design of the pool building and the heating, ventilating and lighting systems. This paper, however, will be limited to a discussion of the hydraulic features, the water purification plant and other facilities for sanitation.

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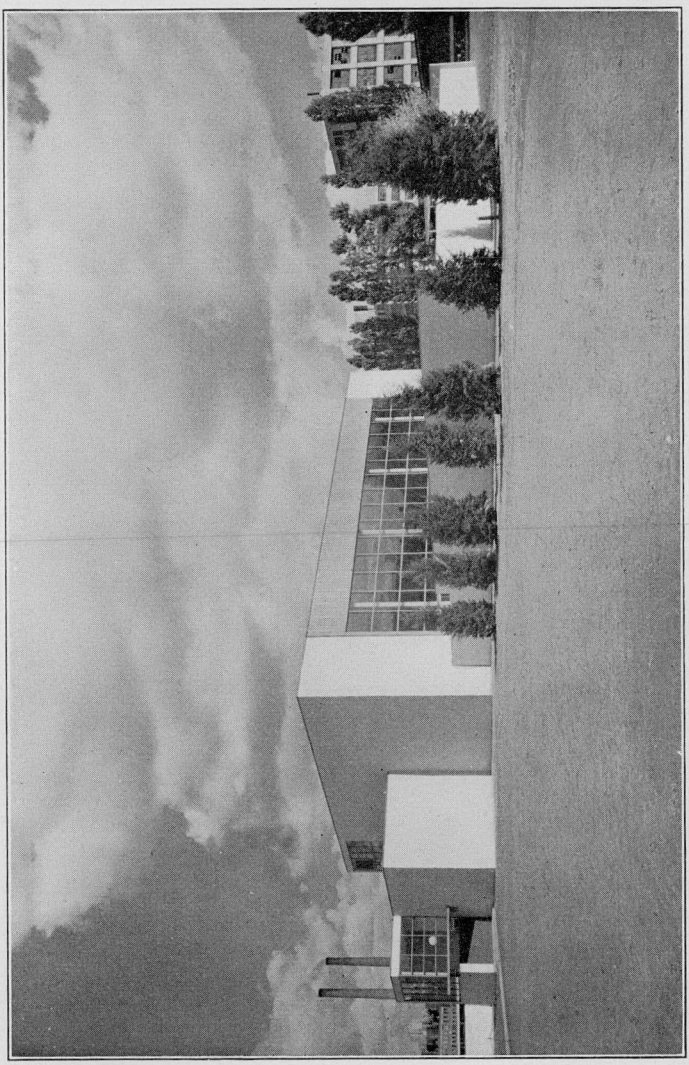
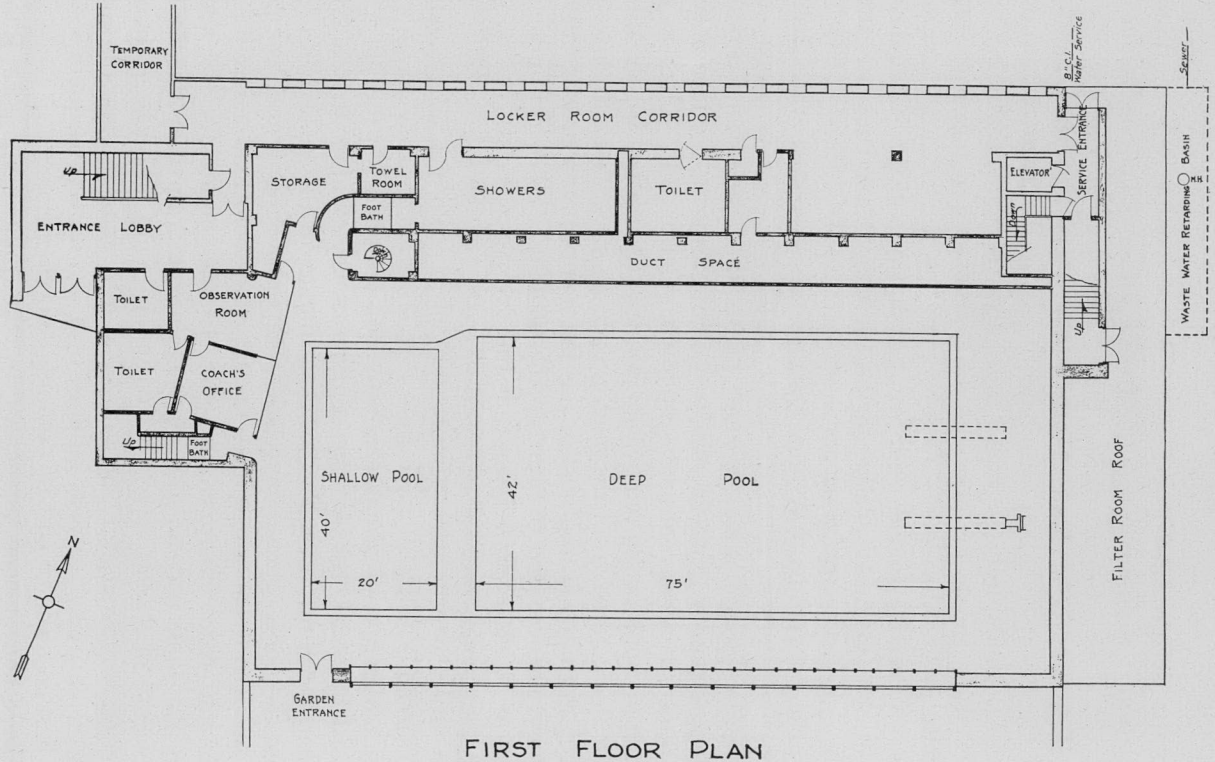


FIG. 1.—ALUMNI POOL FROM THE SOUTHWEST



FIRST FLOOR PLAN

Fig. 2.—First Floor Plan

Basis of Design

In the design of indoor swimming pools employing continuous recirculation it is common practice to provide an 8-hour recirculation period. The 1937 Report of the Joint Committee on Bathing Places of the Conference of State Sanitary Engineers and the American Public Health Association recommends that "the rate of turnover should be 3 times daily." The Report also recommends that 3 or 4 pressure filters should be used with a filtration rate of not over 3 gpm per square foot. Alum and soda ash "pots" or other types of solution feeders are recommended and are usually provided. Coagulation tanks are not specifically called for by the Report and they are rarely used for American swimming pools.

In the conventional system, the chemically treated water passes through the filters in about 3 minutes after the alum is added. An alum dose of ½ gpg, which is much higher than the usual swimming pool dose, requires 15 to 20 minutes' mixing time to produce a good floc. Hence in a conventional system coagulation takes place in the pool, and no appreciable removal of floc is accomplished by the filters until the floc returns from the pool. Since an effective alum dose will appear as turbidity in the pool, it has become common practice to use alum only when needed to clear up the pool, or, if used continuously, to apply it in such small doses that the floc particles are not noticeable in the pool water.

In order to provide continuously a clear, colorless water and to give greater flexibility of operation, a coagulation tank (Fig. 3) with a detention period of about one hour was designed for the M. I. T. pool. Such a tank permits the continuous use of alum in relatively high concentrations, if required, without danger of the passage of much floc into the pool. It was expected by this provision to keep the pool water so free from suspended matter that chlorination would be cheaper and more effective. In order to compensate for the added cost of the tank and for the increase in operation cost due to the expected use of a relatively high continuous alum dose, the recirculation period was fixed at 12 hours instead of 8 hours. Thus the capacity of the entire system was reduced by 1/3 with a consequent lowering in first cost and in cost of pumping and chemicals.

The 8-hour period advocated by the Joint Committee Report

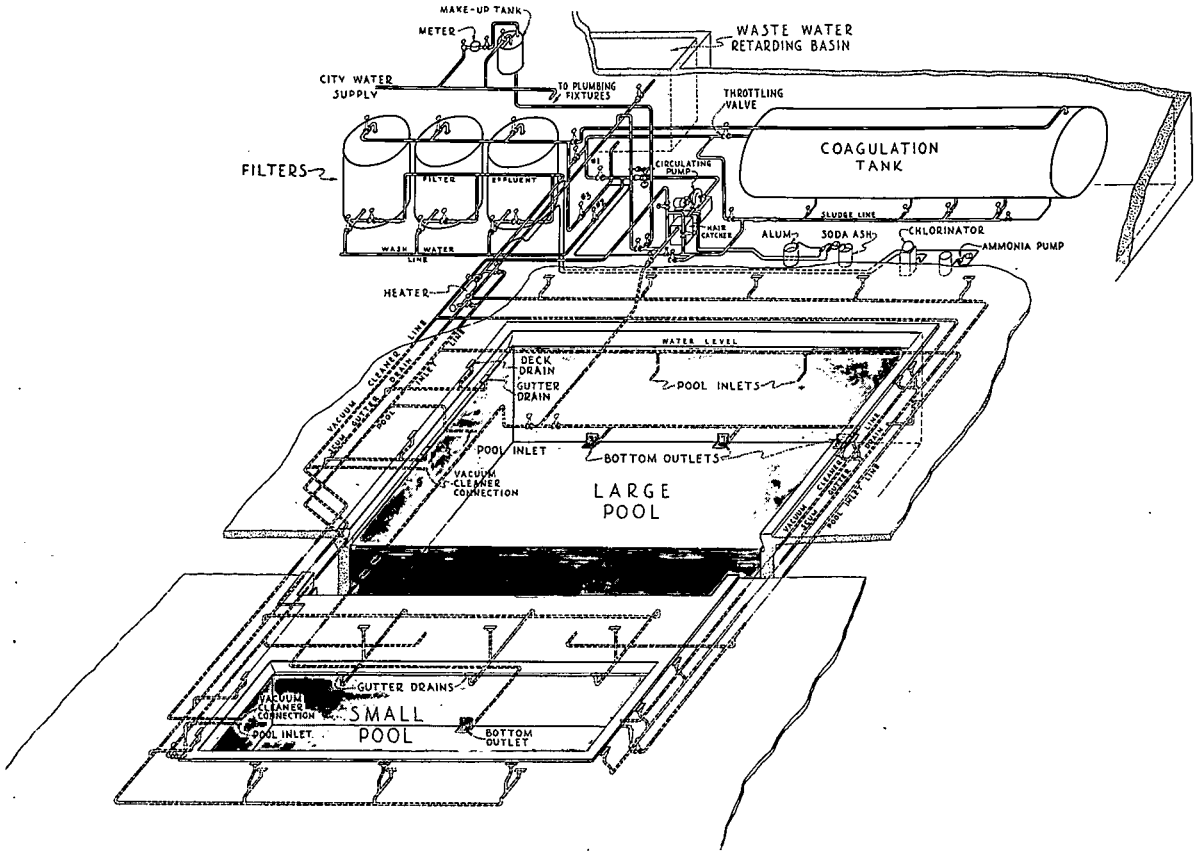


FIG. 3.—RECIRCULATION SYSTEM

is based upon early studies by Gage and Bidwell and the fairly satisfactory performance of conventionally designed systems. Neglecting the dilution from the make-up tank, the change in concentration of dirt in the pool water per day may be represented as follows:

$$\frac{dc}{dt} = L - NRc$$

in which c = the concentration of dirt at the end of t days,
L = the daily load or concentration of dirt added per day,
N = the number of turnovers per day,
and R = the removal efficiency of the filtration system expressed as a ratio.

For most plants with conventional systems, the removal efficiency R is probably not greater than 0.5 and is usually less due to inadequacy of coagulation. It is evident from the form of the equation that equal results will be obtained for the same load for all systems with equal values of N times R. Hence 2 turnovers with an efficiency of 0.75 is equivalent to 3 turnovers and a 50% efficiency. With adequate coagulation made possible with a tank for that purpose, it was expected that an overall removal of 75% of the dirt would be readily obtained. Hence a 12-hour recirculation period with adequate coagulation is considered to be at least as effective as an 8-hour period with no tank.

The proper function of the filters is to remove turbidity and color. Since both must be so low in pools, it is impractical to measure filter efficiency in terms of turbidity or color. A good measure of efficiency of removal may be had with tests for residual alumina, if alum is used as the coagulant. Bacterial removal by the filters is of no moment, since disinfection must be adequate to keep the pool water safe at all times. Chlorination with a sufficiently high dose to keep an adequate residual throughout the pool and the recirculation system seems to be the best method of accomplishing this end. Continuous chlorination was therefore adopted for Alumni Pool, and provisions were made for applying both ammonia and chlorine solutions into the discharge pipe from the filters.

The large pool was made 75 feet long and 42 feet wide to provide six seven-foot swimming lanes. To insure the safety of divers, the depth was fixed at 13 feet at the deep end. The depth at the

shallow end was made 7 feet. Hence bathers cannot stand in this pool, and the heavy bathing loads possible with bathers standing in shallow water are avoided. The small pool is 20 feet wide by 40 feet long with the bottom sloping transversely, the water depth being 3.5 feet at the west side and 4.5 feet at the side adjacent to the shallow end of the large pool. The small pool is thus subject to heavier pollution than the large pool, and while the two pools are supplied from the same recirculation system, provision was made for varying the rate of turnover in the small pool. This was accomplished with two large capacity inlets (1½" diameter pipes), each being provided with a Venturi meter and throttling valve. With these valves wide open the rate of turnover in the small pool is three times the rate for the large pool. The total volume of the two pools is about 280,000 gallons, the capacity of the large pool being about 10 times that of the practice pool.

For a 12-hour turnover period, the recirculation rate should be 388 gpm. The design of the recirculating system was based upon a normal discharge of 360 gpm with the coagulation tank in use, but a rate as high as 540 gpm may be employed without the tank in the circuit.

The surface area of both pools is 3950 square feet. According to the 1937 Report of the Joint Committee this should accommodate a peak load of 146 persons, allowing 27 square feet per person. According to the 1935 Report of the Joint Committee, the rate of recirculation should be at least 1000 gallons in 30 minutes for each 20 persons. For 146 bathers this requirement leads to a minimum discharge rate of 244 gpm. Inasmuch as the peak bathing load for Alumni Pool was estimated at about 75 persons, the normal rate of recirculation is well above the Joint Committee requirements.

It is common practice in the design of swimming pool recirculation systems to use pumping heads from 35 to 50 ft. About 12 ft. (5 lbs. per sq. in.) of this head is allotted to pressure loss through filters, the remainder being distributed between the piping and the inlets. This relatively high loss in pipes and inlets makes it possible to dispense with rate controllers and use the same constant speed centrifugal pump for both recirculation and filter washing. For example, during filtration the variation in pumping head throughout the run will be increased by only 10 to 20% due to filter clogging and

the consequent change in pumping rate is not excessive. Also, since the washing process requires less head, the pump discharge is increased for washing and with three filters it is possible to wash them one at a time with rates up to 24 inches rise per minute.

The above described practice results in savings in the first cost of equipment, but it increases pumping costs. It was estimated that the cost of pumping at Alumni Pool would be $9.20 per year per ft. head at the normal rate of recirculation. At a 6% interest and depreciation rate, the capital value of this power is $153. Hence a saving in pumping head of 20 ft. would justify an additional first cost of equipment up to $3000. In the design of the system for Alumni Pool, every effort was made to keep the pumping head low. Friction losses in piping and inlets were kept to a minimum. A rate controller was provided to control the pumping rate during both filtration and washing, and a variable speed drive was provided for the pump to get wash rates up to 36 inches rise per minute.

Hydraulic Model Studies

A number of model studies were made in an effort to reduce wave action in the large pool to a minimum. The first of these studies, made under the direction of Prof. K. C. Reynolds, was concerned with the profile of the bottom. A 1/8 scale wooden model was built and provided with a false bottom which could be fixed in any desired position. Waves were produced by dropping heavy objects into the water of the model pool and the time rate of damping of these waves was measured. Measurements were made with the bottom in several different positions, and with the water level both below and above the lip of the gutters. No influence of the bottom profile was discernible.

The bottom was then fixed to slope at a steep grade to within 20 ft. of the deep end and thence on a 1% grade to the end wall. The usual procedure of fixing the deepest part of the pool off the ends of the diving boards was not followed in order to avoid the necessity of placing the bottom drain pipes under the concrete floor. The bottom outlets are in the end wall of the pool.

A second series of experiments was made on full size sections of scum gutters of various types to determine the effect of gutter shape upon wave action in the pool. A number of different designs

were constructed in wood in 6 ft. lengths. Each model was placed across one end of a shallow concrete basin which was equipped at the other end with a mechanical wave-making machine. The models were tested with the water level below the lip of the gutter, at the lip of the gutter and 5 inches above the lip. These experiments indicated that waves were most severe with straight vertical walls and least objectionable with the scum gutters submerged. It was found that louvres placed in front of the gutters were effective in damping waves. No substantial difference in the damping effectiveness of gutters of various shapes was found. For the safety of the bathers standing on the curb, the gutter finally selected was so shaped that the edge of the curb is flush with the face of the pool wall below the gutter.

Computations of pipe friction losses indicated that for recirculation of pool water by way of the scum gutters a difference in water level in the gutters on the two sides of the large pool of about 3 inches would obtain. Hence the gutters were made 5 inches deep for the large pool and 3 inches deep for the small pool. The gutter inverts were made level to facilitate the pacing of the tile. Gutters were placed only on the side of the pools, the end walls being left blank to facilitate turning of the swimmers in races.

The 1/8 scale model of the large pool was also used to study the design of the inlets. It was desired to keep the head loss at the inlets as low as feasible to conserve pumping head. The minimum loss was limited by requirements of uniform distribution of the filtered water to the pool and by the desire to obtain quick mixing of freshly chlorinated water throughout the pool. Computations of pipe friction losses in the distribution system indicated that a head loss of about 2 ft. through the inlet branches (including the inlet velocity head) was adequate to insure no more than 10% variation in discharge among the inlets. There was some question, however, whether the corresponding inlet velocity of about 8 fps was adequate for dispersion of freshly chlorinated water. It was also considered desirable to place the inlet jets 5 or 6 feet under the water surface to avoid disturbing the swimmers, provided this could be done without unduly lowering the chlorine content at the water surface.

It was tentatively decided to use 14-1 inch pipe inlets for the large pool, 5 on each side at 15-ft intervals and 2 on each end. The

model pool was equipped with 2 sets of model inlets, one set placed 2 feet below the water surface and the other 5 ft. below the surface. Runs were made with a continuous flow of water through the pool at a discharge rate in accord with Froude's model law to correspond with a 13-hour recirculation period in the swimming pool. The first runs were made with the upper set of inlets and the bottom outlets. The remaining runs were made with the lower set of inlets, using first the scum gutters and then the bottom outlets for the pool discharge.

The chlorine dose was simulated by a slug of water containing $KMnO_4$ dye. All inlets were connected by rubber hose lines of equal length to a central dosing chamber through which the inlet water flowed continuously at the normal rate of recirculation. For each run a 100 ml dose of 0.2 N $KMnO_4$ was poured into the dosing chamber. The time of first appearance of the dye at the inlet jets was recorded as zero time. At short intervals of time thereafter, samples of water were collected from the pool in Nessler tubes for comparison with standards as to strength of color. Four sampling stations were used, and at each station samples were taken from the surface, mid-depth and the bottom of the pool.

Fig. 4 shows the rate of dispersion of the dye at the surface of the pool for the three methods of recirculation. These results indicate that freshly chlorinated water will reach all parts of the pool within an hour, and that the concentration of fresh chlorine will be distributed uniformly throughout the pool in about 3 hours. As an adequate residual of chlorine must be maintained in the pool for a recirculation period of 13 hours, all three systems seem to be adequate.

The results at the surface, as shown in Fig. 4, and from the samples taken below the surface show little difference between the three methods of recirculation as regards chlorine distribution. With deep inlets the chlorine concentration at the surface will be slightly less than for the shallow inlets. Chlorine reaches the corners at the deep end of the pool last, and the concentration at the deep end is somewhat less than the average. Recirculation via the bottom outlets improves the chlorine concentration at the deep end.

As a result of these studies 1-inch pipe inlets were adopted for the large pool, and they were placed 5 ft. 6 in. below the water surface.

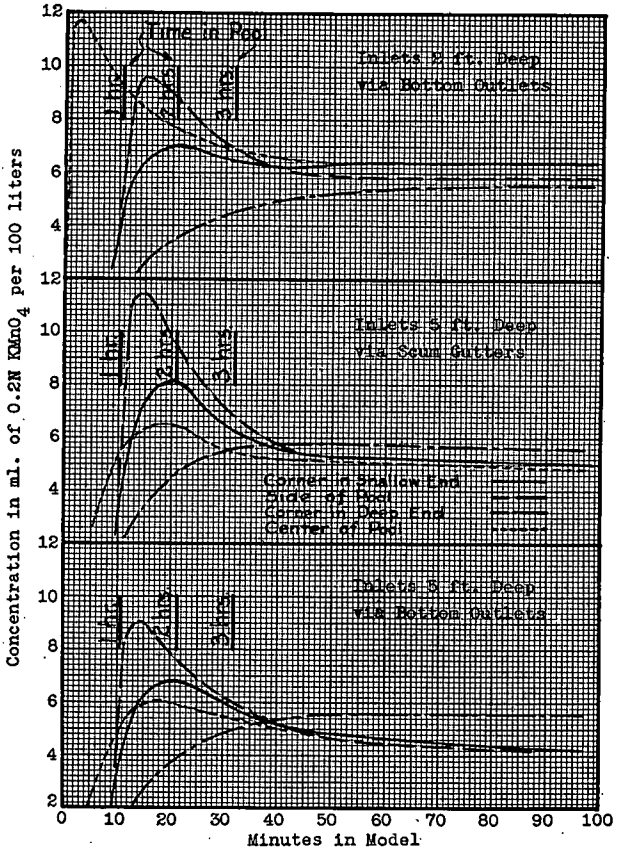


FIG. 4.—DYE CONCENTRATION—TIME STUDIES AT POOL SURFACE

THE RECIRCULATION SYSTEM

The system was designed to permit withdrawal of pool water by way of the bottom outlets or the scum gutters or both (see Fig. 3). When recirculating via the bottom outlets, the scum gutter wash may be sent either to the sewers or back into the filters. In order to save piping, the deck drains discharge into the scum gutter collecting system. Hence when the deck is being scrubbed, this system is opened to the sewers.

During recirculation the pool water passes through the bottom drains or scum gutter drains to a junction point in the pump suction

pipe. Make-up water from the Cambridge system is added at this same point. The water than passes through a hair-catcher to the pump. Alum and soda ash solutions are pumped into the pump suction pipe between the hair-catcher and pump. The pump discharges through a rate controller into the coagulation tank. The effluent from the coagulation tank passes to the filters. The effluent from each filter passes through a Venturi meter provided with a mercury monometer to indicate the rate of flow. The discharge from the filters is then collected in a common pipe into which ammonia and chlorine solutions are pumped. The water passes thence through a heater to the pool inlets.

When the filters need washing, recirculation from the scum gutters is cut off and the pool water is taken from the bottom outlets. Just prior to washing, Valve No. 1 (Fig. 3) is closed slowly. The rate controller and the pump speed are then adjusted for the desired wash rate. Valves No. 2 and No. 3 are opened and the effluent valves on all the filters are closed. The filters are then washed, one at a time, by operating the wash water valves at each filter. The rate of opening these valves is regulated manually by the operator who watches the sand expansion indicators. It will be noted that the filter influent pipe is also used for the waste line. Only three valves are used for each filter. All the valves used in normal operation are controlled from the floor.

Because of the limited capacity of the Cambridge sewer in Vassar Street, it was necessary to provide a retarding basin for the waste wash water. The basin has a maximum water depth of 4.25 ft. and a capacity of 7000 gallons, sufficient storage for about half the water required at the maximum rate of wash.

For cleaning the walls and floor of the pools, 2 - 2″ suction cleaner connections were provided on each side of the large pool and one on each end of the practice pool. These wall fittings discharge to a 3″ suction cleaner pipe line which enters the main suction pipe of the circulating pump just upstream from the hair catcher.

The make-up tank is a cylindrical steel tank 2′-9″ in diameter by 5′-3″ high. It is installed through the main floor with the top of the tank 2 ft. above the floor and about 3 ft. above the normal water level in the pools. The tank is divided into two separate compartments by a watertight vertical steel diaphragm. One compart-

ment is used for the make-up water and the other for a float which actuates a motor operated valve on the make-up pipe.

The water service is an 8″ C.I. pipe entering the building from Vassar Street. This is reduced to 6″ within the building as far as the make-up tank and 4″ beyond to the plumbing fixtures. Make-up water is normally supplied by a 2½″ branch line which is equipped with an angle valve for throttling, a 2″ compound meter and a 2½″ motor operated gate valve. This pipe discharges freely over the top of the make-up tank. For quick filling of the pool a 6″ auxiliary line is provided with a manually operated gate valve. The float well for the automatic valve is connected directly to the large pool by a ½″ copper pipe. A ¼″ lowering of the pool level will open the valve. All water for the normal operation of the pool and filtration plant, except that used for flushing the deck, enters through the make-up tank and is accounted for by the 2″ meter.

The main pipe lines in the recirculation system are 8-inch, the velocity being 2.3 fps during recirculation at 360 gpm and 6.5 fps when washing at the peak rate of 1000 gpm. The total estimated pumping head during recirculation at 360 gpm is about 19 ft., including a 12 ft. allowance for filter loss of head with the rate controller wide open. Hence the loss of head in the piping is about 7 ft., of which 1.3 ft. is in the suction pipe and the remainder on the discharge side of the pump. The loss through the inlet branches is about 2.1 ft. During washing at the peak rate of 1000 gpm, the total estimated head loss is 17.8 ft. including 3.7 ft. in the filter underdrains and 2.3 ft. through the gravel and sand. Since there is a drop of 2.8 ft. between the pool level and the maximum water level in the retarding basin, the maximum net pumping head required for washing is 15 ft.

All piping of larger diameter than 2 inches is of steel with cast iron flanged joints, the flanges being threaded on the pipe. Fittings and gate valves are flanged cast iron, valves being bronze mounted. All steel pipe and tanks are lined on the interior with Bitumastic Enamel for protection against corrosion. Fittings and valves are unlined and will be replaced when badly corroded. All piping 2 inches in diameter and smaller is of copper with bronze fittings and bronze gate valves. Since it was expected to operate with a pH value around 7 and a high chlorine residual both conditions being favorable to corrosion of iron, every precaution was taken to minimize corrosion.

Joints between iron and copper have been insulated to minimize electrolysis.

The hair-catcher is an 8-inch Type HH Josam with 8-mesh heavy bronze wire screen, the inside of the body being lined with Bitumastic Enamel. It is equipped with a differential mercury manometer for indicating loss of head due to clogging of the strainer.

The pumping unit consists of a horizontal, split casing, single stage, double suction centrifugal pump connected through a V-belt variable speed drive to a 7.5 H.P. squirrel cage induction motor. The pump and motor were furnished by Fairbanks-Morse and Co., and the variable speed drive was furnished by U. S. Electrical Motors, Inc. Pressure gauges are provided on the pump suction and discharge pipes. The pump efficiency is about 80% at the design rate of recirculation, 360 gpm against a 19 ft. head; the corresponding brake horsepower being about 2.2. The speed required is approximately 920 R.P.M. The pump will deliver the peak wash rate of 1000 gpm against a 15 ft. head with a speed of about 1300 R.P.M., the brake horsepower being about 7.5. Typical characteristic curves are shown in Fig. 5.

The coagulation tank (Fig. 6) is a cylindrical steel tank 9 ft. in diameter by 40 ft. long. The cylindrical shell is of 5/16 in. steel and the ends are of ⅜ in. steel plate dished to a 9 ft. radius. The

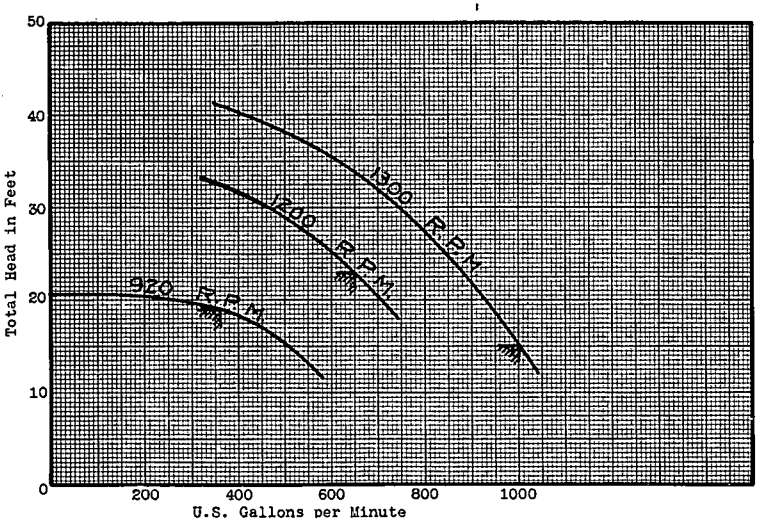


FIG. 5.—CHARACTERISTIC CURVES FOR CIRCULATION PUMP

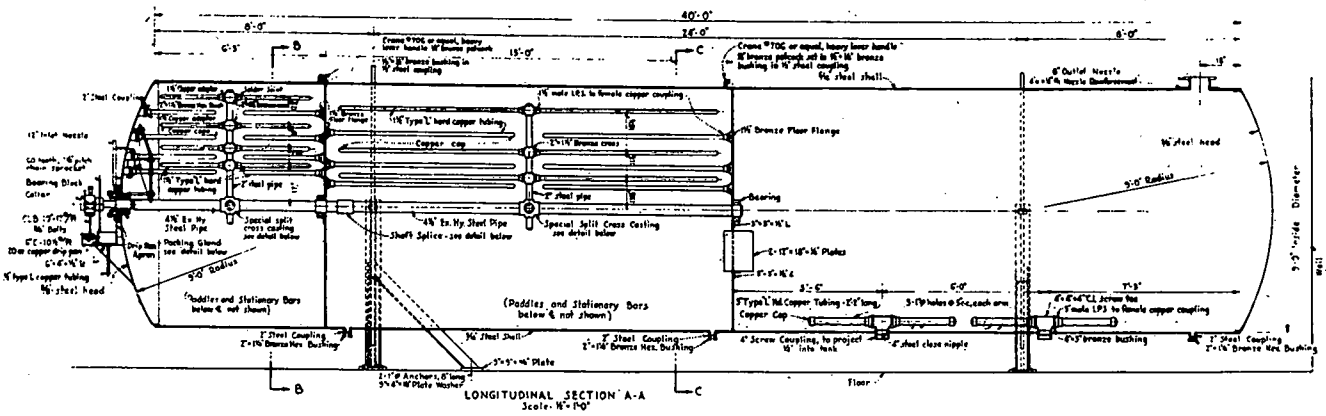


FIG. 6.—COAGULATION TANK

capacity of the tank is about 19,600 gallons, which provides a detention period of 54.5 minutes at the 360 gpm rate of recirculation. The tank was designed to withstand a test pressure of 50 lbs. per sq. in., and will be operated under a pressure of less than 15 lbs. per sq. in. The method of support, designed by Prof. John B. Wilbur, is novel for water tanks. Collars consisting of steel rings 1 in. by 7 in. in section are welded around the tank 8 ft. from the ends. The tank is supported at 4 points on pins which pierce these collars on the sides of the tank and which are in turn supported by steel columns. The construction is welded throughout.

The interior of the tank is divided into three interconnected compartments by steel diaphragms. The first two compartments, 6 ft. 3 in. and 15 ft. long respectively, are equipped with a stirring mechanism for coagulation. The mechanism consists of a 4½ in. steel pipe shaft on the tank axis fitted with cross arms in the middle of each compartment. Each arm in the first compartment supports 4 revolving longitudinal rods and each arm in the second compartment supports 3 revolving longitudinal rods. Stationary longitudinal rods are provided between each two revolving rods, and are fixed to the diaphragms and end wall in rows at 120° intervals. The mechanism provides a somewhat more rapid mix in the first compartment than in the second at the same speed of rotation. The mechanism is driven by a Reeves 3 H.P. Motodrive at speeds from 2 to 10 R.P.M.

The third compartment in the tank was designed for settling, in order to reduce the load on the filters if heavy alum doses should be required. Provision was made for recirculating the sludge from the bottom of this compartment to the inlet end of the tank. It was hoped thereby to get the effect of a heavy floc with a smaller alum dose.

The method used for recirculation is believed to be novel. (Fig. 3.) A 4-in. line was provided from the bottom of the sludge compartment to a point in the inlet pipe to the tank just upstream from a throttling gate valve. At this point the 4-in. line is fitted with a 2½-in. nozzle which enters the inlet pipe and is bent to discharge axially at the lower part of the throttling gate valve. Partial closing of the throttling valve lowers the pressure at the end of the 2½-in. nozzle and produces flow through the sludge line. A 4-in. by 2-in. Venturi meter with a carbon tetrachloride manometer was provided in this line for measuring the rate of return of sludge. Computations indi-

cate that to produce a rate of recirculation of 25 gpm, the velocity through the throttling valve must be about 6 fps. The maximum velocity in the sludge line at this rate is 2.5 fps at the throat of the sludge meter.

The filters are of the pressure type (see Fig. 7) with vertical

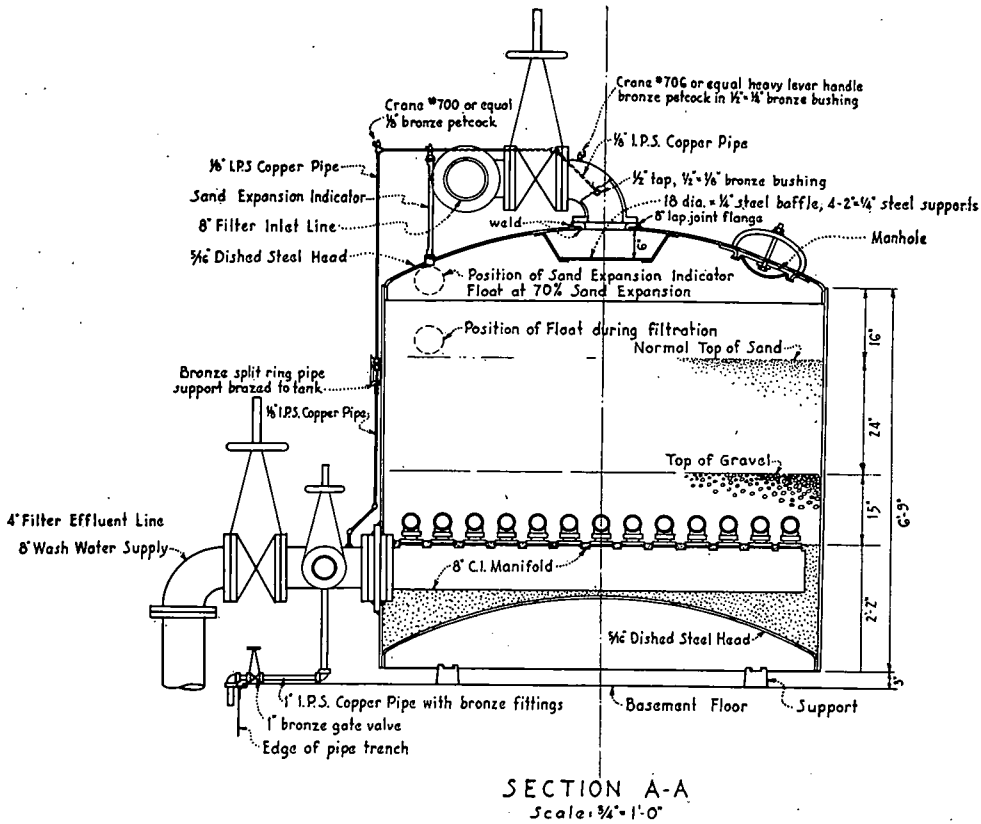


FIG. 7.—DETAIL OF FILTERS

steel cylinders 7 ft. 6 in. in diameter and having a side wall height of 6 ft. 9 in. Both top and bottom heads are dished upwards to a radius of 7 ft. 6 in. The sides are of 1/4-in. plate and the heads of 5/16-in. plate, all-welded construction. The tanks were designed for a test pressure of 50 lbs per sq. in. Each filter underdrain consists of an

8-in. C.I. manifold and 2-in. copper pipe laterals spaced 6½-in. on centers. Each lateral is perforated on the under side with ⅝-in. round holes spaced 7 in. on centers, the orifice ratio being 0.6%. The coarse gravel, 2 to 2½-in. in size is supported on bricks which are placed on edge between the pipes. The gravel is graded in layers down to a small size of 10-mesh to ¼-in. at the top. The gravel supports a bed of sand 24 inches deep.

The filters are designed for a normal rate of filtration at 360 gpm of 2.7 gpm per sq. ft. The maximum wash rate, 1000 gpm, is equivalent to a 36 in. rise per minute. Ample height is provided in the shells for a 70% sand expansion.

The filter sand is from Cape May, N. J., and has a mechanical analysis as shown in Fig. 8. Expansion tests were made on this sand

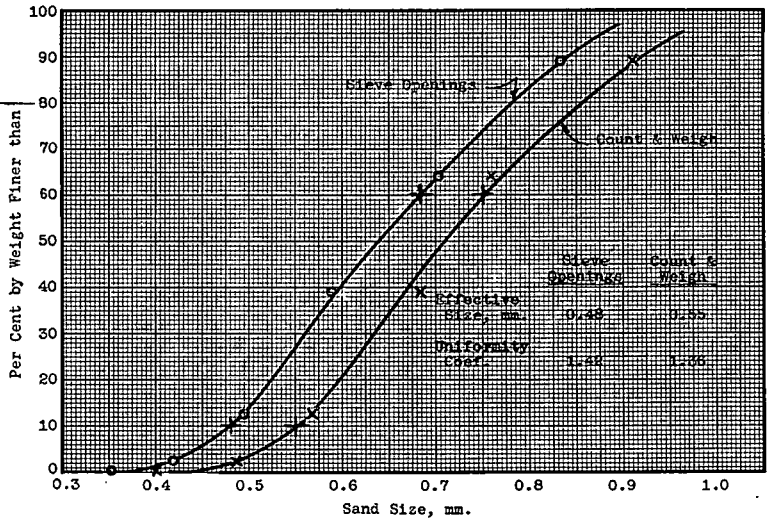


FIG. 8.—SIEVE ANALYSIS CURVES FOR FILTER SAND

in the laboratory to furnish data for the calibration of the sand expansion indicators and for selecting the proper wash rates. The results of these tests are shown in Fig. 9. It will be noted that the maximum wash rate provides an expansion of 39% and is just adequate to get all the sand in suspension. Washing is normally done at a 20% expansion with about 55% of the bed in suspension.

Alum, soda ash and ammonia (Fig. 3) are prepared as strong

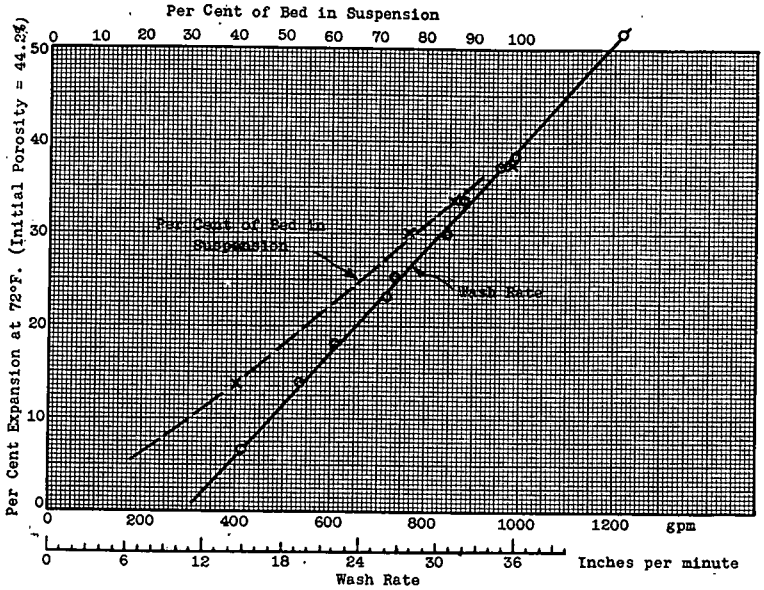


FIG. 9.—SAND EXPANSION CURVES

solutions and pumped with Proportioneers diaphragm feed pumps to the points of application. Ammonia is purchased as aqua ammonia in 13-gallon carboys and diluted with pool water in a fully enclosed solution system. Chlorine is purchased as liquid chlorine in 150 lb. drums and fed as a solution with a Wallace and Tiernan Type MSP-M chlorinator. Solution water is obtained from the pool inlet line and boosted to the required injector pressure by means of a small gear pump. All solution lines are flexible rubber-lined hose. The chlorinator and ammonia pump are housed in an enclosed room, which is provided with forced draft ventilation from floor and ceiling.

Alumni Pool was constructed by the Aberthaw Company of Boston as general contractor. The filtration and recirculation system was installed by Wm. H. Mitchell & Son Co. of Boston as the plumbing subcontractor. The system cost approximately $24,000 with all accessories and equipment. The filters, coagulation tank and appurtenances were fabricated by the Graver Tank & Mfg. Co., Inc., of East Chicago, Indiana. The rate controller, Venturi meters and gauges were furnished by the Builders Iron Foundry of Providence,

R. I. The design for the filtration and recirculation system was prepared by the author with the able assistance of Dr. P. C. Stein.

Operating Results

Recirculation was started at Alumni Pool early in August and the pool was officially opened for use August 26, 1940.

Operation was started at a rate of 360 gpm and with an alum dose of ½ gpg. As a result of jar studies for optimum pH made in the Sanitary Engineering Laboratory, the soda ash dose was adjusted to maintain the pH at about 7.5. The chlorine dose was adjusted at the start to maintain a residual of about 0.5 ppm in the pool, and no ammonia was used. It was decided to use a 10% solution of sodium thiosulphate in the foot baths instead of the usual hypochlorite solution in order to avoid the strong chlorine odor characteristic of hypochlorite foot baths. Since the deck drains discharge into the scum gutter drain pipe, it was decided to allow the scum gutter wash to discharge into the sewer in order to preclude excessive reduction of the chlorine in the pool by the thiosulphate tracked in on the feet of the bathers.

During the succeeding weeks of operation a number of changes have been made in the procedure used at the start. As a result of analyses and other measurements, operating technique has been greatly improved. A number of the changes which have been made and the reasons for making them are described below.

It was soon found that the waste to the sewer by way of the scum gutters amounted to from 30,000 to 50,000 gallons per day. Since this amount was quite variable, moreover, close control of pH and chlorine residual was out of the question. To prevent this excessive waste of water, it was necessary to return the scum gutter wash to the system. The extra chlorine dose which it was expected might be required to offset the effect of the thiosulphate did not materialize. In fact, the chlorine dose has been less rather than more, since the excessive dilution of pool water was stopped. The pool is now operated at a rate of 330 gpm, recirculating via the scum gutters during the day and via the bottom outlets at night. The turnover period is about 15 hours for the large pool and 9 hours for the small pool.

It was hoped that with an alum dose of ½ gpg sufficient sludge

could be developed in the settling tank to permit sludge recirculation after a few weeks. After the pool water was cleared of the rust and sediment brought in from the city mains during the original filling, it was found that the floc particles were so light that very little settling occurred in the tank. Laboratory jar tests were made with heavier doses of alum up to 1.5 gpg and with equivalent doses of ferric sulphate in an effort to find a floc which would settle effectively. In all cases, however, the floc was too light for settling. In order to reduce the load on the filters, it was then decided to reduce the alum dose to the smallest which would produce a floc before the water reached the filters.

Recent laboratory studies indicate that a dose of 1/6 gpg requires about 1½ hours mixing before a visible floc is produced. Experience at the plant, however, indicates that the particles are large enough with one hour's coagulation to be removed effectively by the filters. A residual of about 0.05 ppm of aluminum can be maintained in the pool with an alum dose of 1/6 gpg. This indicates a removal by the filters of about 80% in terms of aluminum. On the other hand an alum dose of only ⅛ gpg resulted in a residual in the pool of about 0.15 ppm of aluminum which corresponds with a filter efficiency of about 55%. The optimum pH of the pool water for an alum dose of 1/6 gpg now in use is about 7.0. Water from the bottom of the sludge compartment is being recirculated at 20 gpm to furnish nuclei for the treated water in an effort to speed up flocculation. The stirring mechanism in the tank is being operated at a speed of 4 R.P.M.

It was found that a sufficient chlorine residual for effective germicidal action could not be maintained with chlorine alone without unpleasant odors and complaints from the swimmers. Ammonia was therefore applied with the chlorine, first with a ratio of one ammonia to five of chlorine and later with a ratio of 1 to 10. It has been found that a chlorine residual of 0.8 ppm is more than adequate for disinfection with the 1 to 10 ratio and that a residual of 0.5 ppm is too low. At present the chlorine residual is maintained at about 0.7 ppm. For more than a month the plate counts on nutrient agar after 24 and 48 hours have been zero, with very few exceptions. Positive tests for coli have been very rare. A chlorine dose of about 0.4 ppm appears adequate to maintain a residual of 0.8 ppm.

The air temperature in the pool room is maintained at about

85°F. It is planned to keep the pool temperature between 75° and 80°F. Thus far it has not been necessary to heat the pool water, except for a short period during September. The temperature remains at about 77°F.

The color of the pool water has been a matter of considerable interest. When the pool was first filled the water contained sediment and rust. It was quite turbid and was deep green in color. When the water was cleared up so that the turbidity was reduced to about 0.5 ppm the color changed to a beautiful sky blue. With continued improvement the turbidity has been reduced to zero, as read by a Hellige turbidimeter. The color, however, has reverted to green. No effort has been made to explain this phenomenon, but as a matter of interest Prof. Arthur C. Hardy made an analysis of the color of the pool water in the Color Measurements Laboratory. This analysis together with comparative studies of distilled water and tap water are shown in Fig. 10. No measurements of color according to "Standard Methods" have been attempted on the pool water.

Filter runs average about 5 days with an alum dose of 1/6 gpg. The average amount of water used per wash has been about 9000 gallons for the three filters. This amounts to about 0.4% of the water filtered. With the gutter wash returned to the system, the daily use of water for purposes other than washing has been about 400 gallons.

The hair catcher is cleaned every day. The friction loss through the basket at the normal rate of recirculation is about 0.2 inches of mercury when the hair catcher is clean. At the end of a day's run the friction loss is 2 to 3 inches of mercury.

The number of bathers within the pool room is counted every half hour, and the total number of men who enter the pool daily is counted by means of a photoelectric cell installed in the corridor between the footbath and pool. The number of swimmers per day has averaged between 250 and 300, with an average swimming time per bather of about 20 minutes. The average number of persons in the pool room at a time has been about 8, and the peak number so far noted was 67.

On week days, the pool is open to bathers from 11 A. M. to 10:30 P. M. On Sundays and holidays the hours are from 2 to 9:30

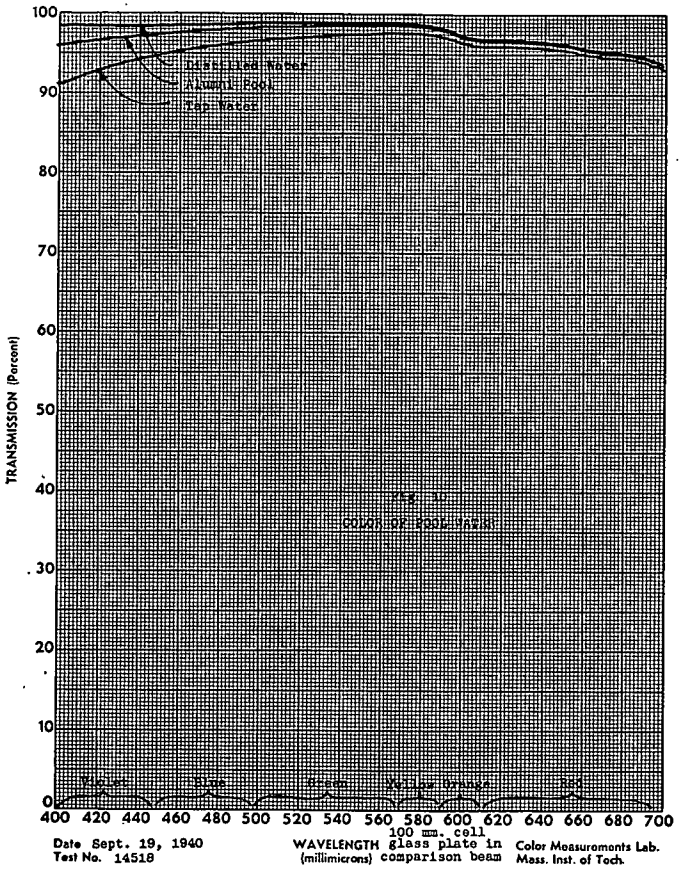


FIG. 10.—COLOR OF POOL WATER

P. M., and mixed bathing is permitted. On Monday evenings the pool is reserved for women between 6:30 and 10:30 P. M.

No clothing is worn in the pool except during meets and when women are permitted to use the pool. All trunks and women's suits are owned by M. I. T. and issued with the towels. Bathrobes and paper sandals are also issued by the matron to women bathers. Rubber bathing caps for women and for men who desire them are available for purchase.

Tests for pH and chlorine residual are made twice each day by the operator on samples from each pool and from the filter efflu-

ent pipe. Tests for turbidity are also made twice a day on the pool water. Residual aluminum tests are made every few days on the pool water. Samples for bacterial analysis are taken twice per day on Mondays and Wednesdays. In the morning one sample is taken from each pool; and in the afternoon three samples are taken from the large pool, one from the small pool, and one each from the pump suction pipe and the filter effluent pipe. On each of these samples total counts are made on nutrient agar plates after 24 and 48 hours' incubation at 37°C. Each sample is also examined for gas formers in 10 ml. portions in lactose broth at 37°C. after 24 and 48 hours, and confirmations for E. coli are run on any tubes which show gas.

At intervals of 2 weeks, analyses are also made on the pool water for iron, total solids, loss on ignition, sulphates and oxygen consumed. Typical chemical analyses are shown below which indicate changes in the pool water over a ten weeks' interval.

	Sept. 4	Nov. 15
Iron, ppm	0.9	0.2
Total solids, ppm	289	291
Loss on ignition, ppm	0.6	56
O_2 consumed, ppm	1.9	5.0
Sulphates, ppm	57	86

Bacteriological and chemical analyses are made in the laboratories of the Department of Biology and Public Health.

Alumni Pool is operated by the staff of Mr. Albert V. Smith, Superintendent of Buildings and Power. Prof. Carl M. F. Peterson, Assistant Superintendent, has direct supervision of operation. The full time personnel consists of a filter operator whose hours are from 9 to 6; two deck attendants who work in two shifts, from 7 to 3 and from 3 to 11 respectively; and two attendants who divide their attention between the pool and Barbour Field House and who work in two shifts from 7 to 3 and from 3 to 11 respectively. Periodic trips are also made to the pool by mechanics from the heating and ventilating service. The pool decks are washed and the foot baths are changed every morning. The floor and walls of the pools are cleaned with the vacuum cleaner on Tuesdays and Fridays.

The sanitary control of the pool is under the direction of a Sanitation Committee of which the author is chairman. The other members of the committee are Prof. Murray P. Horwood, Prof. John W. Williams and Prof. Marshall W. Jennison.

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING

Boston Society of Civil Engineers

NOVEMBER 20, 1940.—A joint meeting of the Boston Society of Civil Engineers and the Hydraulics Section, B. S. C. E. was held this evening at the Engineers Club and was called to order at 7:10 P.M., by Vice-President Albert Haertlein. One hundred ten members and guests attended the meeting. Ninety-three persons attended the dinner before the meeting.

Vice-President Haertlein announced the deaths of the following members:

Robert B. Farwell, who died May 7, 1940, and had been a member since January 27, 1915.

Charles E. Wells, who died August 15, 1940, and had been a member since February 17, 1897.

Henry C. Hartwell, who died October 27, 1940, and had been a member since January 10, 1875.

Arthur L. Plimpton, who died July 6, 1939, and had been a member since February 17, 1898.

The Secretary announced the election of the following members on September 18, 1940:

Grade of Member—*Harry S. Perdikis, Rutger E. Peterson.

Grade of Junior—Giles L. Evans, Jr., †Harold E. Sanford.

Grade of Student—Roger A. Brown, Richard A. Morse, Hugh J. Noonan, Thomas Prizio, William E. Savage, Jr., Thomas J. Tevlin.

Vice-President Haertlein requested Vice-Chairman Scott Keith of the Hydraulics Section to preside during the brief business meeting of the Hydraulics Section, relative to the appointment of a Nominating Committee for that Section.

Vice-President Haertlein then introduced the speakers of the evening, who gave talks on the general subject—"Geological Exploration for Dam Sites."

Speaker—F. Stewart Brown, Chief of the Flood Control Design Section of the U. S. Engineer Office in Boston.

Subject—"Foundation Investigations for the Franklin Falls Dam."

Speaker—Frank E. Fahlquist, Senior Geologist of the U. S. Engineer Office of Providence, Rhode Island.

Subject—"New Methods and Technique in Sub-Surface Explorations."

Speaker—Irving B. Crosby, Consulting Geologist.

Subject—"Objectives and Methods of Geological Investigations of Dam Sites."

The talks were illustrated bv lantern slides.

*Transfer from Grade of Junior.

†Transfer from Grade of Student.

Following the talks, a number of members and guests took part in discussion and the question period.

Adjourned at 9:40 P.M.

EVERETT N. HUTCHINS, *Secretary.*

DECEMBER 18, 1940.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the Engineers Club. Sixty members and guests attended; fifty-six persons attended the dinner.

The Secretary reported the election of the following to membership on December 18, 1940:

Grade of Member—Earl R. Baker, Frank Cheney, Langley S. Homer, Martin A. Mason, Gordon S. Rutherford, P. Charles Stein, *W. Robert Williams.

Grade of Junior—†Harry L. Freeman.

President Shaw read a recommendation of the Board of Government at a meeting held on November 20, 1940, that an amendment to the By-Laws be passed relative to the use of the current income of the Permanent Fund, and stated that the notice sent to members of the Society announced that this matter would be considered at this meeting. This recommendation provided that the Board of Government may use the whole or part of the annual income of the Permanent Fund for current expenses in any fiscal year, authorization for which for a number of years has been voted each year by the Society.

On motion duly made and seconded, *VOTED* that Section 9 of the By-Laws be stricken out and the following inserted in place thereof. "Section 9. Permanent Fund:—There shall be a fund called the Permanent Fund, to which shall be added all money received for entrance fees and all income from investments of the fund. In any fiscal year of the Society the income from the Permanent Fund, may, upon vote of the Board of Government, be transferred to the current fund for the payment of current expenses to the extent necessary to maintain the current fund. No other sum shall be appropriated from the Permanent Fund except by a two-thirds vote at two successive regular meetings of the Society; and any such appropriation or any part thereof not used within three years shall be returned to the fund."

The President stated that this matter will be acted upon at the next regular meeting of the Society to be held on January 8, 1941, the date for which was set by the Board of Government at the meeting on November 20, 1940.

President Shaw then introduced the speaker of the evening, Mr. Harry T. Carroll, Resident Engineer, Boston Transit Department, who gave a talk on "The Huntington Avenue Subway."

At the conclusion of the paper, a question period brought out interesting additional data about the subway.

The meeting adourned at 8:45 P.M.

EVERETT N. HUTCHINS, *Secretary.*

JANUARY 8, 1941.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the Engineers Club and was called to order by President Arthur L. Shaw. The President stated that the Board of Government had voted on November 20, 1940 to hold the regular meeting of the Society on this date instead of the usual fourth Wednesday. He stated also that this is a Joint Meeting with the Designers Section.

The President announced the death of Charles R. Felton, who had been a member since October 19, 1892 and died November 22, 1939.

The President stated that at the last meeting of the Society, held on December 18, 1940, a vote was passed to amend Section 9 of the By-Laws, relative to the use of the income of the Permanent Fund. The announcement of this meeting stated that this matter will be acted upon at this time. The President read

*Transfer from Grade of Junior.
†Transfer from Grade of Student.

the proposed amendment as voted upon at the last meeting. Upon motion duly made and seconded *VOTED* that Section 9 of the By-Laws be amended by striking out said Section and inserting in place thereof the following: "Section 9. Permanent Fund: There shall be a fund called the Permanent Fund, to which shall be added all money received for entrance fees and all income from investments of the fund. In any fiscal year of the Society the income from the Permanent Fund, may upon vote of the Board of Government, be transferred to the current fund for the payment of current expenses to the extent necessary to maintain the current fund. No other sum shall be appropriated from the Permanent Fund except by a two-thirds vote at two successive regular meetings of the Society; and any such appropriation or any part thereof not used within three years shall be returned to the fund."

The President called upon Mr. Kimball R. Garland, Chairman of the De signers Section, who stated that the Designers Section was pleased to join with the main Society in this meeting.

President Shaw then introduced the speaker, Dr. Charles Terzaghi, who gave a paper on "The Critical Load on Strata of Clay beneath Foundations," illustrated by lantern slides.

Ninety-eight persons attended the meeting and seventy-three attended the dinner.

Adjourned at 9:15 P.M.

EVERETT N. HUTCHINS, *Secretary.*

SANITARY SECTION

MARCH 6, 1940.—A regular meeting of the Sanitary Section was held this evening at the Society Rooms, 715 Tremont Temple. Prior to the meeting thirty-four members and guests attended the dinner at Patten's Restaurant. The meeting was called to order by Ralph M. Soule, Chairman, with fifty present.

The clerk read the report of the Executive Committee for the year ending March 6, 1940, and it was voted that the report be accepted and placed on file. The report of the nominating committee was read by Mr. Flood, and it was voted that the clerk be authorized to cast one ballot for the nominees as read. Accordingly, the following officers were elected for the year 1940-41: *Chairman,* C. Frederick Joy, Jr.; *Vice-Chairman,* Edwin B. Cobb; *Clerk,* George B. Coffin; *Executive Committee,* Ralph M. Soule, Professor Charles O. Baird, Walter E. Merrill.

The matter of forming a committee to study the question of "Limiting Velocities of Flow in Sewers" was presented and Chairman Soule was authorized to appoint such a committee, and accordingly the following men were appointed to serve on this committee: Frank L. Flood, Chairman, George F. Brosseau, Thomas R. Camp, Herman G. Dresser, James F. Folsom, William F. Haley, Ralph F. Horne, Paul F. Howard, Walter E. Merrill, George A. Sampson.

The speaker of the evening, Earle B. Phelps, Professor of Sanitary Science, of the College of Physicians and Surgeons, Columbia University, gave a talk on "Air Sanitation." Professor Phelps discussed the results of studies which he has made relative to Air Sanitation and pointed some of the harmful effects of smoke, dust and industrial gases. Professor Phelps presented his subject most interestingly and there was much interest in the discussion which followed.

Adjourned about 9:00 P.M.

GEORGE W. COFFIN, *Clerk.*

MAY 1, 1940.—A regular meeting of the Sanitary Section was held this evening at the Society Rooms, 715 Tremont Temple, and was called to order at 7:00 P.M. by the Chairman, C. Frederick Joy, Jr. Eighty-one members and guests attended the meeting, and forty persons had dinner at Patten's Restaurant, Court Street, prior to the meeting.

VOTED to join the Main Society on their excursion in Boston Harbor in

June, and not have a separate Section outing.

The Board of Government of the B. S. C. E., having authorized the formation of a new section to be known as the Hydraulics Section, Chairman Joy offered the organizing committee for the new section the privilege of using this meeting as a business meeting for the formation of the new section. Accordingly, the meeting was turned over to H. B. Kinnison, who read the report of the organizing committee, which report, will appear in the minutes of the Clerk of the newly formed Hydraulics Section. After the election of officers of the new section, Chairman Joy invited all those present who had come to organize the new section to stay and hear the paper and discussion at the Santitary Section meeting, which invitation was accepted by everyone present.

The Speaker of the evening was, William L. Hyland, Assistant Engineer, Fay, Spofford & Thorndike, Consulting Engineers, who presented a paper on "Asbestos—Cement Pipe for Water Mains and Sewers."

Fay, Spofford & Thorndike have been in charge of the design and supervision of the construction of the new Cranston, Rhode Island, sewerage system and have used in this system about two miles of 14" to 20" asbestos cement pipe for gravity sewers and one mile of 6" to 10" force main. Mr. Hyland discussed the design and hydraulic works field. His paper which was carefully prepared and very interestingly presented was illustrated by slides, showing details of construction and cost analyses comparing this pipe with Vitrified Clay Pipe. At the conclusion of Mr. Hyland's paper there was an interesting discussion at the end of which a rising vote of thanks was given Mr. Hyland for his paper.

Adjourned about 9:30 P.M.

GEORGE W. COFFIN, *Clerk.*

OCTOBER 2, 1940.—A regular meeting of the Sanitary Section was held this evening at the Society Rooms, 715 Tremont Temple, with thirty-nine members and guests present. Thirty persons attended the dinner at Patten's Restaurant prior to the meeting. The meeting was called to order by the Chairman, C. Frederick Joy, Jr.

Walter E. Merrill, Secretary of the Committee on Limiting Velocities of Flow in Sewers, presented a preliminary report of that committee.

The Chairman introduced the Speaker, Roy S. Lanphear, Supervising Chemist of the Bureau of Sewers of the Department of Public Works, Worcester, Mass., who gave a paper on "Operations of the Worcester Sewage Treatment Works."

Mr. Lanphear who has been in charge of the Worcester Sewage Treatment Plants since its construction during the years 1919 to 1925, described the works and discussed the various processes in the treatment of the sewage. He also spoke of some of the difficulties encountered in operating a large treatment works in an inland city and described methods used for overcoming these difficulties. His talk was very interestingly presented and a long discussion and question period followed.

Meeting adjourned about 9:30 P.M.

GEORGE W. COFFIN, *Clerk.*

DECEMBER 4, 1940.—A regular meeting of the Sanitary Section was held this evening in Walker Memorial, M.I.T., Cambridge, and was called to order by the Chairman, C. Frederick Joy, Jr. Sixty-five members and guests attended this meeting and thirty-two persons attended the supper prior to the meeting.

On motion duly made and seconded *VOTED* to authorize the Chairman to appoint a nominating committee to nominate officers for the year 1941-1942. Accordingly, Chairman Joy nominated the following to serve on this committee—Paul F. Howard, Richard S. Holmgren and Samuel M. Ellsworth.

The chairman introduced the speaker, Thomas R. Camp, Professor of Sanitary Engineering, Massachusetts Institute of Technology, who gave an interesting illustrated paper on "The Filtration System for the new M.I.T. Swimming Pool—Design and Operation".

The new Technology pool is, undoubtedly, the most up to date swimming pool in the country today, and was designed and constructed as an all Technology enterprise. Over 50 members of the faculty worked on various features of the design. The filtration system was designed and constructed under the direction of Professor Camp. His paper was very interestingly presented and contained much valuable information for designers and operators of swimming pools. There was a discussion period of interest. At the conclusion of the meeting, 8:30, a visit was made to the swimming pool where ample opportunity was afforded for the inspection of the filtration system as well as the pool itself. Several of the members availed themselves of the invitation to take a swim in the pool.

GEORGE W. COFFIN, *Clerk.*

DESIGNERS' SECTION

OCTOBER 9, 1940.—A regular meeting of the Designers Section was held this evening at the Society Rooms, 715 Tremont Temple, attended by sixty members and guests and was called to order at 7:00 P.M. by the Chairman, Kimball R. Garland. The dinner at the Ambassador Restaurant, prior to the meeting was attended by thirty persons.

The minutes of the previous meeting were read and approved.

The Chairman then introduced Mr. Frank J. Crandall, Assistant Chief Enginner of the Liberty Mutual Insurance Company who spoke on the subject of "Failures and Their Causes." The subject aroused lively interest, and following an active discussion the meeting adjourned at 8:45 P.M.

EMIL A. GRAMSTORFF, *Clerk.*

DECEMBER 11, 1940.—The regular December meeting of the Designers' Section was held as a joint meeting with the Highway Section this evening at the Society Rooms.

The meeting was called to order by Chairman Donald P. Taylor of the Highway Section at 7:10 P.M. Following the transaction of some business of that section, Mr. Taylor turned the meeting over to Chairman Kimball R. Garland of the Designers' Section who presented the speaker, Mr. L. G. Sumner, Engineer of Bridges and Structures of the Connecticut State Highway Department.

Mr. Sumner spoke on "The Merritt Parkway and the Housatonic River Bridge." The talk covered the development of the modern highway in a comprehensive manner and through the use of many fine colored slides showed the incorporation of these developments in the highway under discussion. The speaker also outlined and discussed the major features of the river crossing. The many fine slides depicting the entire project were prepared and furnished by the Bureau for Street Traffic Research at Yale University.

After some discussion, the meeting adjourned with a rising vote of thanks to Mr. Sumner for his kindness in coming to Boston to deliver this talk.

The meeting adjourned at 8:25 P.M. The attendance was forty-three.

EMIL A. GRAMSTORFF, *Clerk.*

NOVEMBER 13, 1940.—A joint meeting of the Sanitary and Designers' Section was held this evening at the Society Rooms and was called to order by Chairman, Kimball R. Garland of the Designers' Section at 6:40 P.M.

Following the reading and acceptance of the minutes of the previous meeting, the chairman introduced Mr. C. Frederick Joy, Chairman of the Sanitary Section, who extended an invitation to the designers to attend the next meeting of their section to be held at the Massachu-

setts Institute of Technology on December 4, 1940.

The Speaker of the evening, Mr. Chester J. Ginder, Assistant Civil Engineer, Metropolitan District Water Supply Commission, was introduced by the Chairman and he presented an interesting paper on the "Design of Fabricated Plate Steel Tees, Laterals, and Wyes of Large Diameters for the Pressure Aqueduct of the Boston Metropolitan District Water Supply Commission."

Mr. Edwin B. Cobb, co-author of the paper and formerly Senior Civil Engineer Draftsman for the Commission, offered some additional comments.

The meeting adjourned at 8:15 P.M. Attendance, thirty.

EMIL A. GRAMSTORFF, *Clerk.*

HIGHWAY SECTION

APRIL 24, 1940.—A regular meeting of the Highway Section of the Boston Society of Civil Engineers was held this evening in the Society rooms, 715 Tremont Temple.

The meeting was called to order by the Chairman, Mr. Donald W. Taylor, at 7:05 P.M.

The minutes of the previous meeting were read and approved.

Mr. Taylor thanked the members of the Section for their confidence in electing him chairman and pledged his efforts to the success of the coming year.

Mr. Taylor then introduced Mr. Dwight M. McCracken, Director of the Traffic and Safety Bureau of the Liberty Mutual Insurance Co., who spoke on "Methods and Uses of a Traffic Safety Survey."

Mr. McCracken stated that the purpose of the surveys is to prevent death, injury and economic loss to citizens.

He outlined the various steps and studies in the surveys and illustrated his talk by charts, diagrams and "before and after" photographs which attested to the efficacy of the recommendations made as a result of the surveys.

A question period followed.

A rising vote of thanks was given Mr. McCracken for his excellent talk.

There were eighteen members and guests present.

The meeting adjourned at 9:00 P.M.

LOUIS H. SMITH, *Clerk.*

HYDRAULICS SECTION

Organized May 1, 1940

The organization of the Hydraulics Section was provided in order to occupy a field not previously covered in the Section activities of the Society and as a result of the wide interest shown in the series of lectures designated as the John R. Freeman Lectures on Hydraulics, which were proposed by Prof. Gordon M. Fair, President, 1939-40, with the cooperation of the John R. Freeman Fund Committee. The Board of Government voted on March 20, 1940, to authorize the establishment of a new section to be known as the Hydraulics Section, Boston Society of Civil Engineers. On April 11, 1940, the Board of Government designated a committee, consisting of Karl R. Kennison, Chairman, Harvey B. Kinnison, and Arthur L. Shaw, to inaugurate the steps necessary to organize this new section.

On May 1, 1940, the meeting for the organization of the Hydraulics Section was held at 715 Tremont Temple, Boston. Mr. H. B. Kinnison served as temporary Chairman of this meeting for the adoption of By-Laws for the Section and for the election of officers.

The following were elected officers for the current year:

Chairman, Donald F. Horton; *Vice-Chairman,* Scott Keith; *Clerk,* M. T. Thomson; *Executive Committee,* Stanley M. Dore, Dr. Kenneth C. Reynolds, Dr. Harold A. Thomas.

Mr. Frank B. Walker, President, B. S. C. E., on behalf of the members of the

Board of Government extended congratulations and best wishes to the new section, the fifth section to be formed in the Society. He referred briefly to the formation of the first four Sections whose By-Laws were approved by the Board of Government as follows: Sanitary Section, January 27, 1904; Designers' Section, May 19, 1920; Northeastern University Section, November 23, 1922; and Highway Section, May 21, 1924.

Through the courtesy of the Sanitary Section, Mr. Frederick C. Joy, Jr., Chairman, the new section was invited to join with the Sanitary Section, whose meeting was held on this same date, to hear a paper by Mr. William L. Hyland, Assistant Engineer, Fay, Spofford and Thorndike, on "Asbestos-Cement Pipe for Water Mains and Sewers."

Eighty-one persons attended this meeting.

M. T. Thomson, *Clerk.*

NORTHEASTERN UNIVERSITY SECTION

September 27, 1940.—The first meeting during the school year 1940-1941 of the Northeastern University Section B. S. C. E. was held today in room 440-W. Twelve attended the dinner which was held at the Lobster Claw at 6:00 P.M.

President Walter B. Kelley opened the meeting at 7:15 P.M., at which thirty-two were present. Numerous older members of both the Boston Society and the American Society of Civil Engineers, as well as students from engineering courses other than Civil, were present, indicating widespread interest in the meeting. Mr. Holcombe J. Brown, President of the Engineering Societies of New England attended the meeting.

A letter was read from the Boston Society of Civil Engineers announcing the coming annual joint student night of the Boston Society and American Society of Civil Engineers. Colonel Charles R. Gow was announced as the speaker for the meeting which will be held at Walker Memorial, M. I. T. The program of the coming year was next discussed. Following this a notice was given that a national meeting of all student chapters of the American Society of Civil Engineers was scheduled to be held in Cincinnati.

An announcement was made that William P. Morse, former City Engineer of Newton, has donated his library of technical literature to the Northeastern University Society Section of the Boston Society of Civil Engineers.

President Kelley introduced the guest speaker, Dean William C. White of the Engineering School at Northeastern. Dean White is a member of the Engineering Societies of New England Committee on Public Relations (a committee appointed to investigate engineering registration) as well as being an executive committee member of the same society.

The subject of the talk by Dean White was "Licensing Engineers." He outlined the history of the movement for licensing of Engineers and surveyors and stated that there are only six other states in the country which are without laws which restrict engineers, namely New Hampshire, Massachusetts, Dakota, Missouri and Delaware.

Dean White expressed his views against the passing of such a law in Massachusetts. Among the points which he brought out was the possibility of political exploitation on the part of a board for licensing engineers and the possibility of the regimentation of engineers, which might result from the passage of such a law, which could hardly protect the public more adequately than the existing building laws.

The talk by Dean White was followed by an informal discussion.

The meeting adjourned at 9:10 P.M.

Irving T. Berkland, *Acting Clerk.*

OCTOBER 16, 1940.—The annual joint meeting of the student chapters of the Boston Society of Civil Engineers and the Northeastern Section of the American Society of Civil Engineers was held this evening at Walker Memorial, M. I. T.

The meeting which was scheduled for 7:00 P.M. was preceded by a dinner at 6:00 P.M., both of which took place in North Hall of Walker Memorial. There were one hundred and sixty students in attendance, sixty-five of which were Northeastern men. The students at Northeastern incidentally, were in the majority last year also. Other colleges represented were Harvard, M. I. T., Tufts, Rhode Island State, University of New Hampshire, Brown and Worcester Polytechnic Institute.

Arthur L. Shaw, President of the Boston Society of Civil Engineers opened the meeting by greeting and introducing the various delegations. Charles W. Banks, President of the Northeastern Section of the American Society of Civil Engineers extended his welcome. President Shaw then introduced Colonel Charles R. Gow, the speaker of the evening.

Colonel Gow, besides being an active soldier in the Spanish-American and World Wars, was president of the Boston Society of Civil Engineers in 1916. He has been an inventor, an executive and a teacher. At present he is serving in the capacity of President of Warren Brothers in Cambridge, Massachusetts.

Colonel Gow's topic, "Engineering Personality as an Aid to Professional Success," concerned the personal relationship between co-operating engineers, and students and engineers. His talk treated of a phase of engineering which is seldom touched by speakers. "It is the leaders who are responsible for progress," said Colonel Gow, "and it is the problem of the educational institutions to sponsor leadership." But education is not the essential element in the quest for success; the two qualities without which a man is hopelessly handicapped are:

1. Common sense—the ability to be governed by facts rather than by emotion—a realist as contracted with an idealist.
2. Sense of humor—not taking things, especially oneself, too seriously.

Colonel Gow stated that self-importance is found amongst unimportant people, which fact he supported with a vivid experience of his own. He used many of his varied experiences to exemplify statements which he wished to emphasize.

Following the meeting which closed at 8:45, the delegations circulated and acquaintances were renewed.

IRVING T. BERKLAND, *Acting Clerk.*

OCTOBER 24, 1940.—A meeting of the Northeastern University Section B. S. C. E. officers and executive committee was held today for the purpose of voting upon an amendment to the constitution of the student chapter of the American Society of Civil Engineers, proposed by the South Carolina State College. The meeting was opened at 12:25 P.M. by Winfield B. Knight, Vice-President of the organization.

The amendment proposed a more active individual student membership in the organization. This is offered in place of the present system in which the College Civil Engineering Society was merely affiliated with the American Society of Civil Engineers and the student affiliated with the college society so that a student is one step removed from the American Society. The South Carolina State College proposes to remove this step. The outstanding arguments set forth by the South Carolina College were that the amendment would accomplish the following:

1. Raise standards of membership.
2. Arouse enthusiasm.
3. Facilitate transfer from a student to a Junior member.

4. Make for a closer relation between the student sections.

Mr. Spence, who acted as advisor at the meeting, raised the question of the probable membership fee for a full fledged student member. It was acknowledged that if the amendment was passed, we would be unable to continue our present relationship with the American Society of Civil Engineers. After considerable discussion, a motion was made by Ralph Crowther that our society go on record as being opposed to the final motion states that the Northeastern University Civil Engineering Society goes on record as being opposed to the proposed change in the student organization, and that our Society would take an active part in discouraging the advancement of the amendment. The motion was seconded and approved by those present.

The meeting adourned at 12:50 P.M.

IRVING T. BERKLAND, *Acting Clerk.*

APPLICATIONS FOR MEMBERSHIP

[January 20, 1941]

The By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of fifteen (15) days from the date given.

For Admission

ALBERT ERNEST ABRUZZESE, Boston, Mass. (b. June 5, 1916, Boston, Mass.) Attended North Carolina State College of Engineering from September, 1934, to June, 1935; Lincoln Technical Institute 1936 to present time. Experience, Rodman, Boston City Planning Board; January, 1937 to October, 1938, Draftsman and Transitman, Boston Consolidated Gas Company; October, 1938 to November, 1938, Transitman, George P. Carver Engineering Company, Boston; January, 1939 to March, 1939, Draftsman, Metcalf & Eddy, Boston; April, 1939 to September, 1939, Rodman, Metcalf & Eddy, Boston; October, 1939 to December, 1939, Transitman & Partyman, John Bowen Company, Boston; December, 1939 to present, Surveyman and Draftsman, George P. Carver Engineering Company, Boston. Refers to *A. Burdoin, H. Brask, P. S. Rice, A. L. Shaw.*

HERBERT JAMES ALBEE, Swampscott, Mass. (b. August 7, 1917, Boston, Mass.) Graduated from Wrentham High School in 1935; entered Wentworth Institute in the fall of same year, taking a two year course in Architectural Construction, graduating June, 1937. Experience: August, 1937, employed by the firm of Cleverdon, Varney and Pike, Consulting Engineers, Boston, Mass. Received certificate of completion of Structural Design and Reinforced Concrete design classes under University Extension. Enrolled in Structural Design course by Wilson Engineering Corporation, April, 1940. At present employed with Cleverdon, Varney and Pike, Consulting Engineers. Refers to *H. S. Cleverdon, F. M. Mahard, W. F. Pike* and *E. A. Varney.*

Transfer from Grade of Student

GEORGE ANTHONY, Boston, Mass. (b. September 17, 1912, Greece). B.S. in Civil Engineering from Northeastern University in June, 1935. Experience, June, 1935 to April, 1937, Instrument man with Mass. Geodetic Survey on local control survey and triangulation. May, 1937 to February, 1939, Office engineer on computation and compilation of stream discharge data with the U. S. Geological Survey. March, 1939 to October, 1939, Junior hydraulic engineer on stream gaging with U. S. Geological Survey. March, 1940 to date Recorder on field maintenance and operation of stream gaging stations with U. S. Geological Survey. Refers to *H. B. Kinnison, C. E. Knox, M. T. Thomson, G. K. Wood.*

JOHN LESLIE BEAN, Ayer, Mass. (b. July 17, 1916, Ayer, Mass.) Graduated from Northeastern University in class of 1940, receiving B.S. Degree in Civil Engineering. Experience: November, 1938 to January, 1939, rodman for W. A. Mason and Company, Cambridge, Mass.; April to June, 1939, Transitman for Hayden, Harding and Buchannan, Boston, Mass.; June-July, 1939, Transitman for W. M. Crocker, Boston, Mass.; September to November, 1939, Transitman for N. Y. N. H. & H. Railroad, Hartford Division; January to April, 1940, Chief of party and Inspector for N. Y. N. H. & H. Railroad, Hartford Division. At present employed as Timekeeper for J. S. Packard Dredging Company on the Dredge, "Lawrence." Refers to *C. O. Baird, C. S. Ell, A. E. Everett, E. A. Gramstorff.*

DANIEL JOSEPH CONLIN, West Roxbury, Mass., (b. July 27, 1916, Roxbury, Mass). Bachelor of Science in Civil Engineering, Northeastern University, class of 1940. Experience, 1934-1940, instrument man, layout man, foreman on concrete and brickwork for John Bowen Company, Boston, Mass., and Thomas O'Connor & Company, Cambridge, Mass. At present with George A. Fuller & Merritt Chapman & Scott Corporation, Quonset Point, Rhode Island, worked as inspector of piles and foundations, at present laying out buildings and frences and part time utilities. Refers to *C. O. Baird, C. S. Ell, A. E. Everett, E. A. Gramstorff.*

JOHN JOSEPH GILL, Brighton, Mass., (b. May 14, 1917, Brighton, Mass.) Graduated from Northeastern University June, 1940, with B.S. Degree in Civil Engineering. Experience: under Cooperative Plan at Northeastern University worked for Massachusetts Department of Public Works on road construction; for Metropolitan District Water Supply Commission at the Quabbin Reservoir Project. May, 1936, through November, 1936, City of Newton, Mass., Engineering Department, as rodman and transitman; September, 1937, to November, 1937; March, 1938 to April, 1938; September, 1938 to November, 1938 and April, 1940 to June, 1940 with Boston Housing Authority on Charlestown & Mission Hill Housing Projects with the Cuyahoga Wrecking Company, Cleveland, Ohio, as head timekeeper, paymaster and inspector. June, 1940 to date with Aberthaw Company, Boston, as an assistant to the estimator and the designing engineer. Refers to *C. O. Baird, C. S. Ell, A. E. Everett, E. A. Gramstorff.*

LOUIS G. REINIGER, Saugus, Mass. (b. May 1, 1917, Boston, Mass.) Graduated from Northeastern University, June, 1940, with B.S. Degree and Honor in Civil Engineering. Chairman of student Civil Engineering Society. Experience: Cooperative work at Northeastern University, November, 1937 to November, 1938; student assistant in the Drawing Department of the University; November, 1938 to January, 1939, with William S. Crocker as a rod-

man, and some dredging inspection; January, 1939 to April, 1940, Massachusetts Land Court, draftsman and calculator. April, 1940 to present, with the Thompson & Lichtner Company, Inc., as Junior Civil Engineer (concrete inspection, plant inspection, materials testing, and report writing). Refers to *C. O. Baird, C. S. Ell, A. E. Everett, E. A. Gramstorff.*

ADDITIONS

Members

EARL R. BAKER, 59 Exeter Street, Wollaston, Mass.

FRANK L. CHENEY, 103 Hillcrest Road, Needham, Mass.

LANGLEY S. HOMER, Turner Const. Company, 80 Newbury Street, Boston.

MARTIN A. MASON, 205 Raymond Street, Chevy Chase, Md.

RUTGER E. PETERSON, 87 Highland Avenue, Mansfield, Mass.

GORDON S. RUTHERFORD, 16 Sagamore Road, Wellesley Hills, Mass.

CHARLES P. STEIN, 135 Oxford Street, Cambridge, Mass.

Juniors

BURRITT F. LEIGHTON, 506 Ash Street, Brockton, Mass.

JOHN H. MANNING, 9 Rockland Street, Newton, Mass.

HAROLD E. SANFORD, U. S. Engineer Office, West Springfield, Mass.

DEATHS

CHARLES R. FELTON.... Nov. 22, 1931
HERBERT C. HARTWELL...Oct. 27, 1940
ARTHUR L. PLIMPTON.....July 6, 1939
CHARLES E. WELLS.....Aug. 15, 1940

ADVERTISEMENTS

The advertising pages of the JOURNAL aim to acquaint readers with Professional and Contracting Services and Sources of Various Supplies and Materials. You would find it of advantage to be represented here.

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BOSTON SOCIETY OF CIVIL ENGINEERS

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Paul F. Howard
Walton H. Sears
Donald W. Taylor
(Term expires March, 1941)

John B. Babcock, 3d
Miles N. Clair
George W. Coffin
J. Donald Mitsch
(Term expires March, 1942)

Harold K. Barrows

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Joseph D. Guertin
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Relations of Sections to Main Society

Kimball R. Garland, *Chairman*

Donald W. Taylor
C. Frederick Joy
Donald F. Horton
L. G. Reiniger

1936 Flood Committee

Howard M. Turner, *Chairman*
(See April, 1936, Journal for Membership of Committee)

Welfare

Ralph W. Horne, *Chairman*

John R. Freeman Fund

Charles T. Main, *Chairman*

Charles M. Allen
Robert Spurr Weston
Howard M. Turner

Desmond FitzGerald Award

Carroll A. Farwell, *Chairman*

James E. Lawrence
Robert Spurr Weston

Sanitary Section Award

Robert Spurr Weston, *Chairman*

John P. Wentworth
Frank L. Flood

Designers Section Award

Carroll A. Farwell, *Chairman*

John C. Moses
Waldo F. Pike

Highway Section Award

James E. Lawrence, *Chairman*

Robert Evans
Arthur E. Harding

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