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The Canadian Engineer

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Mechanical Filtration Plant at Lethbridge, Alberta

New Low Service Pumping Station and Relocation of Intake—Heavy Loadings Placed on Soil Capable of Bearing Scarcely Half Ton Per Square Foot When Dry and Affected Easily By Water—Filter Capacity 3,000,000 Gallons Daily

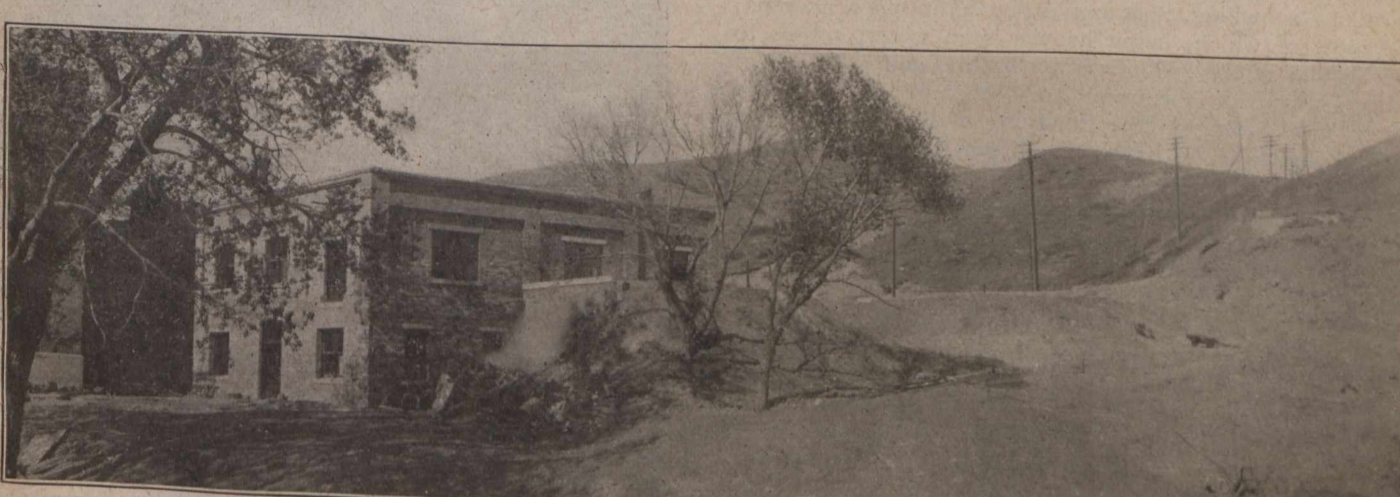
By ARTHUR M. GRACE, M. Am. Soc. C. E.
Formerly Commissioner of Public Works, Lethbridge, Alta.

IN the improvement to the Lethbridge city waterworks, now completed, not only the people of Lethbridge but all persons interested in the clean-water problem should be vitally concerned. The work involved in the present improvements comprises not only the badly needed filtration plant, but also a new low service pumping station for the filters, together with relocation of the intake for facilitating operation of the pumps at low stages of the river.

An item of special interest in connection with the work is the substantial foundation provided for the filter structures, the soil at the site being of an alluvial character, scarcely capable of safely bearing a half ton to the square foot when dry, and easily affected by water, whereas some of the loadings on the columns in the filter structures are as high as seventy-five tons, thus rendering it necessary to construct an elaborate system of sub-foundations, consisting of piling driven to hard-pan and capped with massive concrete pads, safely capable of bearing many times the final load, there being some five hundred piles used in connection with the work.

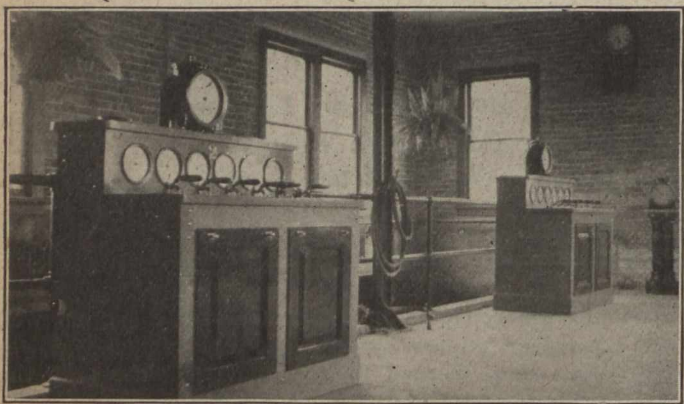
In the operation of the former plant, the intake was located at the shore line of the Old Man River, the water flowing by gravity through a thirty-inch pipe line, approximately 400 feet long, to a suction well alongside the main power house, from whence it was pumped either by a steam pump or by auxiliary electric pumps to the stand-pipes in the city, flowing thence through the distribution mains to the consumers. From this same suction well the condenser pumps serving the lighting and power generators also secured their supply of water; in the new improvements the supply of water necessary for the condensers is obtained directly from the low-lift pumps serving the filters, thus making operation of the condenser pumps unnecessary and effecting a considerable saving in cost of operation.

At low-water periods of the river, considerable difficulty has been experienced in the operation of the pumps, and to overcome this trouble the intake has been extended farther into the river, and also placed at a lower level, so that the water more readily flows to the pump suction well. The two new low-service pumps, one with a capacity of 3,000,000 gallons daily, and the other with a capacity of 1,500,000 gallons daily, are both placed in a pump pit or dry well, some fifteen feet deep, and located under the main filter building, both pumps being of the centrifugal type and especially designed for handling the muddy river water with a minimum amount of wear. The larger pump will be steam-operated, the engine being of the compound type, while the smaller will be electrically operated. Under normal operation only one pump will be in service, the second pump being for reserve or emergency purpose. The chamber in which the pumps are located, being much below the high or flood level of the river, is constructed throughout of concrete, heavily reinforced to withstand the earth and water pressures.



Filter House, Sedimentation Basin and Washwater Tank, Lethbridge Filter Plant

The low-lift pumps take the water from the present suction well and pump same to the inlet chamber of the sedimentation or coagulating basins, the latter being two in number and located at the rear of the filters. These sedimentation basins are provided for the preliminary settling and treatment of the raw water prior to its passing on to the filter beds, and normally will remove from 60 to 65 per cent. of the mud and sediment, it being quite common when cleaning these basins to find three to four



Operating Tables on One Side of Plant

feet thickness of mud in the bottoms of same. Two basins are provided so that one may always be in service when the other is being cleaned; cleaning generally being required every two to five months, depending upon the condition of the raw water. The time required for cleaning, however, is but six to eight hours so each basin is provided with two large blow-off valves, together with flushing lines for hose connections.

The two sedimentation basins have a combined capacity of 500,000 gallons, so that all water passing through these basins normally have a period of four hours settlement before passing on to the filters, the advantage of this longer period of settlement being not only to settle a greater quantity of mud out of the water, but also requiring the use of less coagulant, this resulting in a great saving in the cost of operation.

Coagulation and Sedimentation

As the water passes to the sedimentation basins, a minute quantity of sulphate of alumina as a coagulant is added. The coagulant, as soon as it comes in contact with the natural alkalinity in the raw water, forms a harmless snowlike substance called hydrate of alumina, the most of which, being heavier than water, settles to the bottom of the sedimentation basins, and as it settles collects much of the mud and other impurities in the water, thus aiding in the clarifying of the water. It is desired, however, to have a small portion of the hydrate pass on to the filter with the treated water, where it settles on top of the sand bed, forming a jelly-like coating and together with the sand removes the remainder of the turbidity, and provides a water practically free from bacteria.

In order to insure the full settling capacity of the sedimentation basins, a stilling or diverting wall is placed entirely across these basins at the inlet end, a similar stilling wall being placed at the outlet end, the object of these walls being not only to provide a thorough mixing of the coagulant with the raw water as it enters the basins, but also to insure a lateral distribution of the water across the full width of the basin. Each basin is furnished with sloping floors to facilitate cleaning, an overflow chamber with connection to sewer also being furnished for emergency purposes.

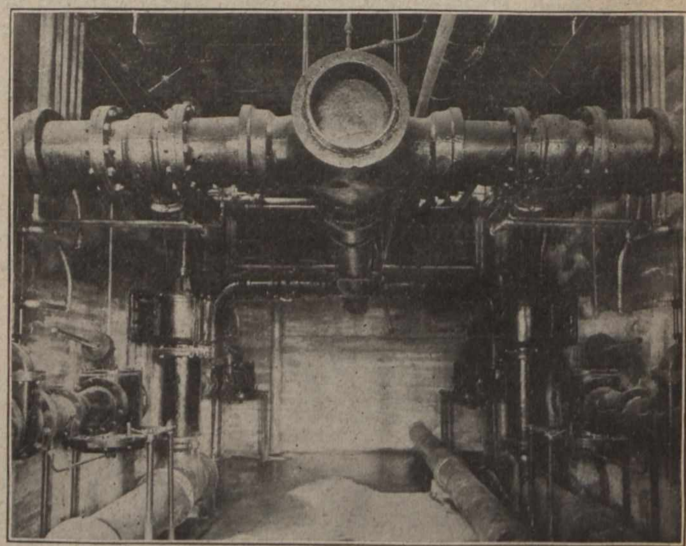
The sedimentation basins occupy a plot about ninety feet square, the depth of water in the basins being about twelve and a half feet. The two basins are constructed entirely of reinforced concrete, the basins also being decked or roofed over with concrete, which in turn is covered with earth as a protection against freezing.

Four Filter Beds

After the settling and treatment in the sedimentation basins. the water flows by gravity on to the filter beds, four in number, and each with a normal capacity of 750,000 gallons daily, or a total of 3,000,000 gallons every twenty-four hours. The filters are arranged in pairs, two on each side of a central operating floor, the valves and other controlling apparatus being located in a pipe gallery underneath this floor. The operating floor as well as the front ends of the four filters are housed over by a brick building, affording ready inspection of the filters at all times, especially desirable when cleaning same. The filters are of the gravity type, each approximately fifteen feet wide by twenty-one feet long, and about seven feet deep. Underneath the four filters and the central pipe gallery is located a reservoir or clear well for receiving the filtered water from the filters and temporarily storing same.

In the bottom of each of the filters, and placed directly on the floor thereof, is located the filter strainer or collecting system. This strainer system is covered by a layer of filter gravel approximately eighteen inches in depth, the gravel in time being covered by the filter sand, a specially selected and uniformly graded material, the depth of sand being about thirty inches.

The strainer system is formed of a grid or manifold of small cast iron pipes, placed at about six-inch centres, and with small bronze strainers located on the under side of the pipes on about 6-inch centres, these strainers form-



Pipe Gallery

ing the collectors for the filtered water after it has passed through the sand and gravel. The strainer pipes or laterals are all connected to larger pipes called manifold headers, which in turn are connected with the outlet or discharge piping leading to the controlling mechanism in the pipe gallery.

On the discharge or filtered water outlet pipe of each of the filters is placed an automatic controlling valve which regulates the rate of filtration from each filter, keeping same constant at any desired rate within certain permissible limits. From the filter rate controllers, pipe connections lead to the clear well, the point of discharge for the filtered water, the high service pumps taking the filtered water from the clear well and pumping same to the standpipe and distribution mains in the city.

Cleaning the Filters

After a period of operation, the length of which depends both upon the turbidity as well as the rate of filtration, the filters require washing or cleaning, and this is accomplished by what is known as the combined air and waterwash system; that is, the use of both compressed air and filtered water through the same distributing system, utilizing the filter strainer system for this purpose. Both are passed in a reversed direction to the direction of filtration, the air first being applied for a period of about 3 minutes, thoroughly loosening up, agitating and scouring the filter sand, the filtered wash water then being applied for 5 or 6 minutes, this forcing up the dirt and sediment to the top of the sand and thence carrying it off by means of centrally located overflow gutters, whence the dirty water passes off into the main filter drain, discharging into the river below the intake.

The air for washing the filters is furnished by an electrically operated Root air blower, at a pressure of about four pounds to the square inch, while the filtered water for washing the filters is obtained from a concrete water storage tank located on the hillside at the rear of the filter plant, the water supply for this tank being secured from the high service force main, an automatic control valve being installed on the supply line to the tank for controlling the water level.

All valves on the piping pertaining to the filters are provided with hydraulic cylinders, so that the entire operation of the filters is controlled from operating tables placed on the floor above the pipe gallery, each filter being furnished with a separate table with individual levers and control cocks for each filter valve. The tables are of polished marble with all fittings and trimmings nickel-plated, each table also being provided, in addition to the valve control handles, with indicator dials for each of the filter valves, these indicators showing whether the valves are open or closed, and if open, the degree thereof. Mounted on each operating table are loss-of-head gauges for showing the operation of the filters. These gauges are provided with electric alarms arranged to notify the operator when the filters become so clogged with mud as to require washing. Gauges are also provided for showing the depth of water in the clear well, also for indicating the water level in the sedimentation basins.

As a further safeguard, previous to passing to the high service pumps and thence to the city, all water is sterilized by liquid chlorine, the apparatus for the application of the chlorine being of the Wallace & Tiernan make. This apparatus is so designed that after it has once been set at a predetermined rate, its action is entirely automatic.

For the preparation and application of the coagulant to the raw water previous to filtration, the entire upper floor of the main filter building is utilized for the storage of the alum as well as for the mixing of same into proper solution, about two-thirds of the upper floor being taken up for storage space, while the remainder is utilized for the dissolving boxes, storage tanks for the coagulant solution, the agitating or stirring apparatus for keeping the solution thoroughly mixed, the scales for making up the solution, etc. For the application of the coagulant to the raw water, very accurate measuring devices are used, the liquid being passed through carefully calibrated orifice feed valves, so that the exact amount of alum used can always be definitely known.

The city commissioners all devoted much time and thought to the water problem, made many personal investigations and visits to various types and makes of plant, and only after long study decided upon the present layout, which they are firmly convinced is of both the most efficient and modern type available, there being but one or two in the Dominion, and only a few in the United States which are at all comparable with it. While the cost is comparatively high, in part due to the very complete equipment, the general high cost of building material, labor and machinery has so radically increased

Power House at Left, Filter House on the Right.

in the past few years, affecting all building work, and the water question was so vital, that it was deemed wiser to proceed with the installation of the plant rather than to wait for lower prices which may not come for years.

Required Elaborate Soil Tests

In view of the city's experience with the faulty foundations of the former power house, much damage having been done to the building through serious settlements and cracking, the commissioners took every possible precaution on the foundations for the filter structure, and required an elaborate series of loading tests on the soil under the filter structures; first, with the object of determining the safe loading capacity of the soil in question; and second, to determine the best type of sub-foundations if same were required. While it was desired if possible, and if it was safe to do so, to avoid the extra cost of this sub-foundation work, the results of the tests of the soil were such as to show the soil to be entirely unsuitable for the loadings of the filter structures, and therefore in order to secure freedom from future settlement and to ensure stability, a comprehensive system of piling and concrete pile caps was provided, amply safe for all future possible loadings.

The general contractors for the filter plant were the Roberts Filter Manufacturing Company, Inc., of Darby, Pa. The sub-contractors for the concrete work and buildings were Smith Bros. & Wilson, Limited, of Lethbridge. The writer found it a pleasure to do business with both the contractor and the sub-contractor on this work which is now complete.

THE ENGINEERING INSTITUTE OF CANADA ELECTIONS AND TRANSFERS

At a meeting of the council of the Engineering Institute of Canada held June 25th, 1918, in Montreal, the following elections and transfers were announced:—

ALEXANDER, KAY, elected member, is a resident of Vancouver, B.C. Major Alexander was born at Scottsville, Ky., in 1875. He received a classical-scientific education at Southwestern University and other noted institutions, and later became actively engaged on various engineering projects. Major Alexander is at present serving in France as second in command, 12th C.R.T.

BONN, WILLIAM ERNEST, of Toronto, was born at Barry Wales, in 1887, and was educated at the Royal Technical College, Glasgow. He has been transferred from junior to associate member. Mr. Bonn, after completing several engineering engagements in Glasgow, became resident engineer of the Canadian Northern Railway Co. in 1911, and three years later became assistant engineer in the Department of Public Works, Canada, on the Toronto harbor improvements.

BRICKENDEN, FREDERICK MALCHORN, of London, Ont., has been transferred from junior to associate member. He was born at London Ont., in 1888 and took a diploma course at Toronto. Mr. Brickenden has had considerable experience in the construction of highways and pavements. In 1915 he enlisted as engineer officer with the Canadian Engineers and is now on active service in France.

BROWN, JOHN ELLIOT, of Ottawa, Ont., has been elected member. He was born at Charlottetown, P.E.I., where he received his education at St. Peter's private school. Since serving apprenticeship with the Royal Electric Co., of Montreal, Mr. Brown has been engaged as engineer in various cities of Canada, and is at present general manager of the Ottawa Hydro-Electric Commission.

BUTTON, STANLEY AMOS, of Virden, Man., has been elected associate member. He was born at Forest Gate, England, in 1881, and received educational certificates from the Board of Education, Kensington, for building construction. After holding several prominent positions in connection with engineering projects in England and also in Canada, Mr. Button was appointed district engineer by the Manitoba government.

COLVOCORESSES, GEORGE M., of Humboldt, Arizona, has been elected member. Born at South Orange, N.J., in 1879, and educated at Yale University, he has, since completing his scientific training, been associated with various engineering enterprises. He is at the present time general manager of the Consolidated Arizona Smelting Co. and consulting engineer for the Ohio Copper Co., of Utah and Nicu Steel Corporation, Limited.

CORRIVEAU, RAOUL DE B., of Ottawa, Ont., was born at West Hoboken, N.J., in 1877. He has been transferred from the class of associate member to that of member. After receiving his degree of B.Sc., at McGill University in 1900 Mr. Corriveau was for several years associated with the Department of Public Works in various capacities. He is at present district engineer, directing investigation and reporting on matters pertaining to the control and improvement of rivers and harbors. He is also making special investigation outside of the district.

HAMILTON, CHESTER B., JR., of Toronto, Ont., is a native of this city and has been elected a member. He received his education at the University of Toronto and after serving apprenticeship in several different shops, principally on engine work, Mr. Hamilton started business on his own account, forming the Hamilton Gear & Machine Co. Since 1915 he has been engaged in the manufacture of shells and other munitions. Mr. Hamilton is a life member of the American Society of Mechanical Engineers, an associate member of the American Institute of Electrical Engineers, and a member of the Society of Automotive Engineers.

KESTER, FRED. HENRY, of Walkerville, Ont., was elected associate member. He was born at Richland, Mich.. in 1885, and was educated as a special student in structural and mechanical engineering at the University of Wisconsin. For the past six years Mr. Kester has been acting as designing engineer in Walkerville, Ont., having full responsibility for designs of steel work for railway and highway bridges, factory and office buildings, etc.

LONGLEY, ERNEST HORACE, of New Westminster, B.C., was born at Barnsley, England, in 1875. Educated at Oxford and Cambridge Universities, he later embarked on a career of constructive engineering and for ten years held the position of chief assistant in the civil engineer's office at Barnsley, where he was in complete charge of all contracts. Mr. Longley is at present chief examiner for the Imperial Ministry of Munitions and has been elected member of the Engineering Institute of Canada.

MARKHAM, ERNEST ARTHUR, of Regina, Sask., was born in 1887 at London, England. He received his education at the Northern Polytechnic Institute of London, and is at present superintendent of construction at Regina. Mr. Markham has been elected associate member.

MOONEY, JOHN PATRICK, of St. John, N.B., has been elected junior. He is a native of St. John and received his degree of B.Sc. at New Brunswick University. Mr. Mooney in 1916 was acting civil engineer and estimator for B. Mooney & Sons, general contractors, brick manufacturers and granite quarry operators. At the present time he is in charge of business in a managerial capacity.

NEILSON, STANLEY ALEXANDER, of Westmount, Que., has been transferred from class of student to that of junior. He was born at St. Thomas, Ont., in 1895, and took a B.Sc. degree at McGill University in 1916. At the present time he is resident engineer of the Hull Electric Co., Hull, Que.

STEWART, ROBERT BRUCE, of New Glasgow, N.S., has been transferred from an associate member to member. He was born at Charlottetown, P.E.I., in 1887 and educated at McGill University. For some years Mr. Stewart was associated with the Dominion Bridge Co. but in 1914 he became manager and engineer of the Maritime Bridge Co., Limited, New Glasgow, N.S.

WALKER, CLAUDE MELVILLE, of Ottawa, Ont., has been elected associate member. He was born at Harriston, Ont., in 1883, and received his degree of B.Sc. at Toronto in 1911 and of A.L.S. in 1914. After holding various responsible positions in the western provinces, Mr. Walker came to Ottawa in 1917 and is at the present time in the Topographical Surveys Branch, Department of Interior.

Proportioning the Materials of Mortars and Concretes by Surface Areas of Aggregates

Proper Mix For Any Desired Strength Can be Predetermined—Results of Tests Show That Strength of Mortars Depends Upon Consistency of Mix and Upon Quantity of Cement in Relation to Surface Areas of Aggregates—Paper Read at Atlantic City Convention of American Society for Testing Materials

By CAPT. LLEWELLYN N. EDWARDS
Supervising Engineer of Bridges, City of Toronto, Ont.

UNDERLYING the combination of inert aggregates with cement to form mortars and concretes, the fundamental principle is that the proportioning of the cement in relation to the aggregates shall be such as to develop the full strength of the inert materials or shall secure conditions of strength, hardness, impermeability or other desired physical properties. While the character of the aggregates and the amount of water used influence results to a marked degree the proper proportioning of the cement is a matter of the utmost consequence, since this is the only active, strength-producing material entering into the mixture.

The "surface-area" method of proportioning assumes as its basic principle that strength, hardness, etc., are primarily dependent upon the relation of the volume of cementing material to the surface areas of the aggregates.

This paper has for its object the following:—

1. To develop information relating to the average surface areas of sand and stone aggregates.

2. To describe the methods and materials used, the results obtained, and the phenomena observed in a series of experimental tests undertaken to develop the practical application and efficacy of this method.

From a careful consideration of the results obtained and of the phenomena observed, the following conclusions appear to be warranted:—

Conclusions

1. The claim that the surface-area method of proportioning the cement content of mortars and of concretes is essentially scientific and rational is proven by the uniformity of results obtained in comparative strength tests of mortars and concretes and by the phenomena observed in these tests.

2. With a given uniform proportion of cement in relation to the surface areas involved, this method provides a definite and practical means of comparing the relative strength-producing qualities of aggregates of varying physical, chemical and mechanical properties.

3. This method provides a means for a more thorough investigation of the functions of sand and stone aggregates and for a more complete development of the true values of all the component materials of mortars and concretes.

4. Excess water in an over-saturated mortar or concrete mix exerts a decidedly weakening effect upon the cement matrix by producing a change in the physical structure of the matrix which tends to destroy the cohesion existing between the particles of cement and the adhesion existing between the cement and the aggregate.

5. In a normal consistency mortar the relation of the area of the particles of the sand aggregate to the cement content of the mix determines the strength of the mortar, provided the strength of the sand material is greater than that of the cement matrix. A similar condition applies to concrete mixes not over-saturated.

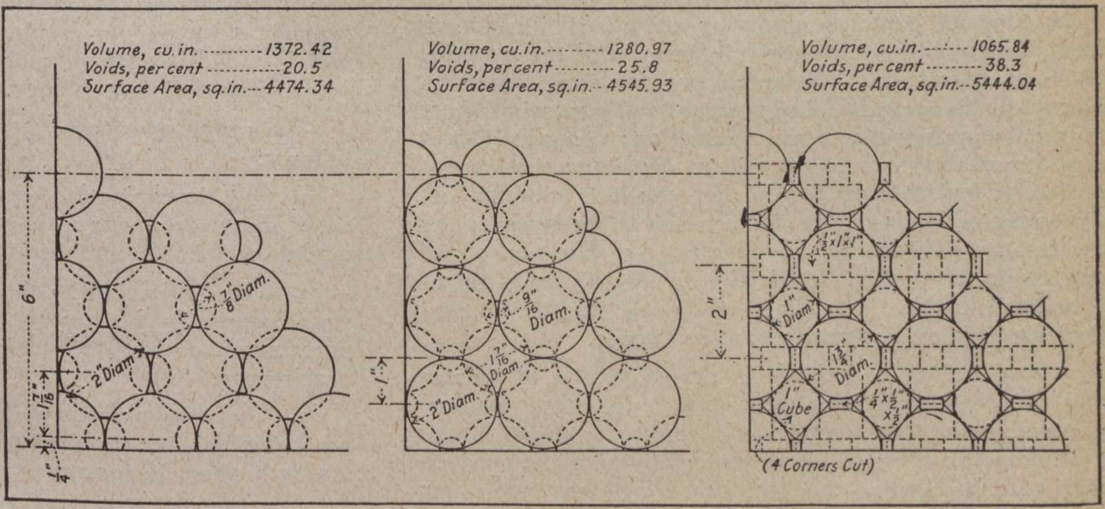


Fig. 1—Relation of Surface Area to Shape of Particles

6. The strengths of mortars containing a given cement and sand but varying for the different mixes in the proportions of these ingredients, are directly proportional to the relation existing between the cement content and the surface area of the aggregate. It follows, therefore, that the strength of a mortar of given mix being known, the mix of a mortar having a desired strength can be pre-determined for the same materials.

7. The quantity of water required to produce "normal," uniform consistency mortars from a given cement combined in varying proportions with sands having the same origin but varying in their granulometric composition, is a function of the water required to reduce the cement to a "normal" paste and the surface area of the sand particles to be wetted.

8. The increase of volume due to the mingling of varying quantities of cement particles with a uniform quantity of a given sand bears no apparent relation to and is evidently not dependent upon the void in the sand.

9. The tests indicate a probable definite relation between the ultimate compressive strength of the mortar content and the ultimate compressive strength of the concrete when the test specimens of the former are produced from mortar of the same mix and consistency as that used in the concrete.

Historical

The use of mortars dates from a time so remote that its beginning cannot be ascertained. However, its modern use in the form of an intimate mixture of lime or cement, sand and water doubtless dates back to the "constructive" Roman period. Both Romans and Normans well understood its use; both had a thorough knowledge of mortar.

Doubtless the method of proportioning the cementing material and aggregates by volume originated among these early artisans. However, the earliest authentic record of its use is credited to General Pasley (1827). This method as commonly specified and used is not only illogical and unscientific, but also unfair to the development of the true value of the materials entering into the composition of both mortar and concrete.

Cement is the binder holding together the aggregates. The fundamental theory of the combination is that the two materials, the one active, the other inert, act as a single unit.

With regard to the item of strength, the ideal mortar contains a proportion of cement sufficient to develop the full strength of the particles of sand aggregate; while the ideal concrete contains a mortar component, in itself ideal, which will develop the full strength of the particles of stone aggregate. The actual proportioning and combining of the cement and aggregates fall somewhat short of reaching this theoretical perfection; practical considerations render it impossible to distribute the cement perfectly, and thus to secure that absolute adhesion between the particles of aggregate which is necessary to the perfect transmission of the stresses from one particle of aggregate to another. Perfection in this regard is therefore unattainable, at least in so far as commonly used field methods and operations are concerned.

Those having experience in practical tests of mortar and concrete know that with given volumetric proportions of materials, comparatively slight variations in the granulometric composition of the sand aggregate produce marked differences in the strength, toughness, and other physical properties of the product.

There should be no place in engineering for guesswork and empiricism whenever scientific determination is possible. With this idea in mind and with a due allowance for the practical considerations mentioned above, it remained to set about the task of finding a structural reason for the wide variations of strength obtained in mortar tests. In November, 1917, a most natural solution presented itself, namely:

1. The strength of mortar is primarily dependent upon the character of the bond existing between the individual particles of the sand aggregate. Upon the total surface area of these particles depends the quantity of cementing material. Furthermore:

2. The amount of water required to produce a "normal," uniform consistency of mortar is a function of the cement and of the surface area of the particles of the sand aggregate to be wetted.

This idea of proportioning the cement in relation to the surface area of the aggregate was new to the writer and appealed to him as a reasonable, although rather radical, departure from the time-honored practice of proportioning by volume measurement. However, he discovered in February, 1918, that it is in reality at least twenty-five years old. Prof. A. H. Heath describes it very clearly in his book "A manual of Lime and Cement," 1893, as here quoted:

> "Sufficient cement should be used to furnish a coating to the surface of each particle of the matrix, and the extent of surface of particles as compared with their bulk is much greater with fine sand than with coarse sand. For instance, a 3-in. sphere has one-eighth the bulk of a 6-in. sphere, but its surface area is one-fourth that of the 6-in.
>
> "To ensure the perfect coating of each particle, a larger proportion of cement must be taken with fine than with coarse sand. If the proportion of cement taken for fine sand be that sufficient only for coarse sand, the cement films enveloping the particles will probably be imperfect, and the concrete will be deficient in strength."

No attempt was made by Professor Heath to develop the surface-area method of proportioning, of which he was a pioneer or possibly the originator.

This paper presents the results of tests made by the city of Toronto Department of Works, under the direct supervision of the writer.

Object and Scope of Tests

The tests were made with the object of developing the surface-area method of proportioning and securing information relative to (1) the surface area of aggregates of varying granulometric composition; (2) the quantity of water necessary to produce a "normal," uniform consistency of mortar for varying sands and cement content; and (3) the strength of mortar attained by varying the proportion of cement in the mix.

The range or scope of the tests was sub-divided as follows:

1. The determination of the average surface area of sand and stone particles of varying sizes based upon an actual count of the number of particles in given weights.

2. The determination, by trial, of the quantity of water necessary to produce, for any given sand and proportion of cement, a mortar of "normal," uniform consistency.

3. The determination of the reliability of the surface-area method of proportioning mortars in so far as strength alone is concerned.

4. The determination of the relative strengths resulting from variations in the cement content of mortars within practical limits.

5. The securing of miscellaneous general information pertaining to the foregoing phases of the tests.

The investigations herein described involve compression and tension tests upon 240 mortar cylinders, 2 ins. in diameter by 4 ins. long; 45 concrete cylinders 6 ins. in diameter by 12 ins long; and 324 standard briquettes.

Surface Area of Aggregates

Possibly the fact that the relation of the surface areas to the volumes of particles of matter depends upon their size and shape needs no demonstration; however, the following example will serve to bring this fact more clearly to mind:

Assume three masses of material, each made up of the following particles:

1. Spheres 2 ins. and 7/8 in. in diameter.

2. Spheres 2 ins., 1 7/16 ins. and 9/16 in. in diameter.

3. Spheres $1\frac{3}{4}$ ins. and 1 in. in diameter, 1-in. cubes, and parallelepipeds $\frac{1}{4}$ by $\frac{1}{2}$ by $\frac{1}{2}$ in. and $\frac{1}{2}$ by 1 by 1 in.

Assume now one cubic foot of each and that the particles are regularly arranged as shown in Fig. 1. Both the total surface area and the total volume in each cubic foot are readily determined. They are as shown in Table I.

Table I.—Relation of Surface Area to Volume of Particles

Cubic Foot No.	Surface Area sq. in.	Volume Cu. in.	Relation of Area to Volume.	Relation to No. 1	
				Area per cent.	Volume per cent.
1.......	4474.3	1372.4	3.26:1	......	......
2.......	4545.9	1281 0	3.55:1	1 6 greater	6.6 less
3.......	5444.0	1065 8	5 11:1	21.9 "	22.3 "

The shapes of particles of sand and of stone aggregates are infinite in number and for this reason no accurate determination of their surface areas is possible. However, it is reasonable to assume that these areas can be closely approximated. Doubtless these approximate areas do not vary greatly from the true areas in the finer particles of sands, since these particles, when examined under the microscope, appear more nearly spherical than do the larger particles of sands and of gravels.

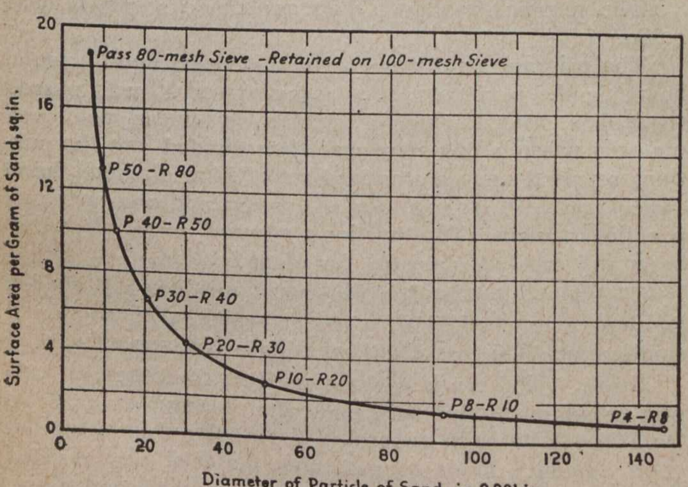


Fig. 2—Surface Area of Sand Aggregate

Surface Area of Sand Aggregate.—As a preparation for the securing of information from which it would be possible to compute the approximate average surface area of sand particles of varying sizes (the exact area cannot be determined) samples of natural sands were secured from different localities. Each sand was thoroughly dried and divided by sieving into its component sizes.

Table II.—Number of Sand Particles per Gram

Name of Sand.	Size of Sieve Passed and Retained on.							
	P4-R8	P8-R10	P10-R20	P20-R30	P30-R40	P40-R50	P50-R80	P80-R100
"Maple" Vaughan, Ont..........	14.75	56.28	403.56	1729	5686	16 891	45 086	72 511
"Pleasant Lake" Otisfield, Me...........	12.60	58.80	40.200	1673	5020	12 183	39 876	98 996
"Cataract" Caledon, Ont..........	10.93	56.17	366.70	1431	4303	15 010	39 510	102 116
"Erin" Erin, Ont.............	13.25	49.80	270 12	1449	4439	17 999	34 542	125 630
"Ontario Lake" Toronto, Ont..........	17.16	57.83	214 00	1409	4584	17 675	40 422	96 412
"Standard Ottawa" Ottawa, Ill............	.....	.....	.....	1323	...	....	....	....
Average................	13.74	55 78	331 28	1502	4806	15 991	40 057	99 133
Assumed Average for Tests	14	55	350	1500	4800	16 000	40 000	99 000

For each sand the average number of particles per gram was determined, for every size of each sand, by an actual count. For the larger sizes 8 to 10 g. or more, and for the medium sizes 3 to 5 g. were counted; while for the smaller sizes 0.25 to 1 g. was counted. The results of these counts, together with the actual and the assumed averages for computation purposes, are given in Table II.

The number of particles per gram having been determined as above described, the specific gravity being 2.689 and the weight of water being assumed at 62.37 lbs. per cubic foot, the volume per particle was computed as follows:

Weight of sand material $= 62.37 \times 2.689 = 167.713$ lb. per cu. ft.
$= 0.09706$ lb. per cu. in. $= 43.0257$ g. per cu. in.

Then

$43.0257 \times$ number of particles per gram = number of particles having a total volume of 1 cu. in.

Whence,

Average volume per particle

$$= \frac{1}{\text{Number of particles having volume of 1 cu. in.}}$$

The average volume per particle being determined, the shape of the particles was assumed as spherical and the surface area per particle and per gram of sand was determined accordingly. The results of these computations are given in Table III. These results, in part, are also shown graphically in Fig. 2. This curve of areas, although empirical, approximates very closely the curve represented by the equation $xy = 139$.

Surface Area of Stone Aggregate.—The shapes of particles of broken stone and of natural gravels are dependent mainly upon the character of the rock material of which they are composed, whether of igneous or of sedimentary origin. The former produce, in broken stone, particles more angular in shape and more rough of

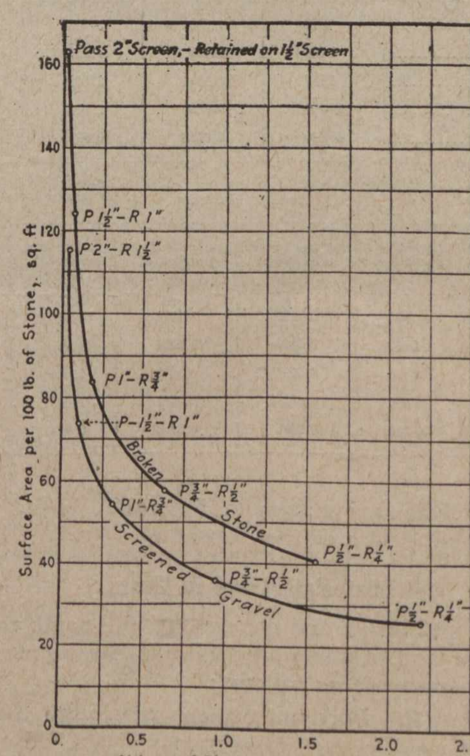


Fig. 3—Surface Area of Stone Aggregate

surface than the latter; while in gravels the tendency of the former is to produce particles approximately more nearly spheres; the particles of the latter are commonly flattened with rounded edges.

The method above described for sand aggregates was followed in the counting of particles, computation of average volumes, etc. In the determination of average surface areas the particles of broken stone were assumed

Table III.—Volume and Surface Area of Sand

Sieve Passed and Retained on.	Number of Particles per Gram.	Average Volume per Particle, cu. in.	Average Diameter per Particle, in.	Surface Area per Particle, sq. in.	Surface Area per Gram, sq. in.
P 4 - R 8	14	0.001 66	0.146 95	0.067 8	0.9492
P 8 - R 10	55	0.000 422 6	0.093 1058	0.027 2335	1.4978
P 10 - R 20	350	0.000 066 45	0.050 2534	0.007 9338	2.7778
P 20 - R 30	1 500	0.000 015 49	0.030 928	0.003 0055	4.5083
P 30 - R 40	4 800	0.000 004 842	0.020 9893	0.001 3841	6.6437
P 40 - R 50	16 000	0.000 001 452	0.014 0818	0.000 6201	9.9216
P 50 - R 80	40 000	0.000 000 5815	0.010 3568	0.000 336 913	13.4765
P 80 - R 100	99 000	0.000 000 2348	0.007 6538	0.000 188 325	18.6442
Standard Ottawa	1 323	0.000 017 57	0.032 2545	0.003 268 38	4.3241

to be made up of one-third cubical and two-thirds parallelepipedal shapes, the latter sub-divided to approximate more closely the areas of the elongated and slab-like shapes of particles. Gravel particles were assumed as spherical in shape. The results obtained for various

Table IV.—Volume and Surface Area of Stone Aggregate

BROKEN STONE.

Screen Passed and Retained on.	Number of Particles in 100 lb.	Average Volume per Particle, cu. in.	Surface Area per Particle, sq. in.	Surface Area per 100 lb., sq. ft.
P 2 - R 1½	670	1 553	8.7047	40.5
P 1½ - R 1	1 680	0.636	4.9261	57 47
P 1 - R ¾	5 680	0 1811	2.1139	83.38
P ¾ - R ½	14 900	0.0692	1.1943	123 58
P ½ - R ¼	43 820	0 0236	0.5339	162 47

SCREENED GRAVEL.

Screen Passed and Retained on.	Number of Particles in 100 lb.	Average Volume per Particle, cu. in.	Average Diameter per Particle, in.	Surface Area per Particle, sq. in.	Surface Area per 100 lb., sq. ft.
P 2 - R 1½	430	2.4289	1.6678	8.7383	26 09
P 1½ - R 1	1 090	0.9509	1 2201	4.6764	35 39
P 1 - R ¾	3 830	0.2694	0.8013	2.0330	54 00
P ¾ - R ½	10 030	0.1020	0.5797	1.0557	73 54
P ½ - R ¼	37 330	0.0276	0.3753	0 4425	114 71

sizes of particles are shown in Table IV. Fig. 3 shows graphically a portion of these results.

Materials Used in Tests

The properties of the cement, of the sand and stone aggregates, and of the water used in the preparation of test specimens were as follows:

Cement.—The Portland cement was, with the exception of that used in the "Consistency Tests," a combination of two brands, namely, "Canada" and "Pyramid." The former was manufactured by the Canada Cement Co., Limited, Montreal, Que., and the latter by the St. Marys Portland Cement Co., Limited, St. Marys, Ont. The two brands were thoroughly mixed by first pouring them together from separate receptacles, then by sieving them several times through a No. 20 sieve.

Following are the physical properties determined in accordance with the Standard Specifications and Tests for Portland Cement of the Society for Testing Materials:

Physical Properties

Constancy of volume		O.K.
Specific gravity		3.12
Initial set, minutes		195
Final set, minutes		450
Fineness.		
Per cent. retained on	No. 100 sieve	3.8
	No. 200 sieve	21.8
Tensile strength, lb. per sq. in.		
Neat	24 hours	312
	7 days	677
	28 days	707
Mortar	7 days	265
	28 days	388

Sand.—All sand used in the tests was of limestone origin, and with the exception of that in one concrete test was secured from the Lake Shore Sand & Gravel Co., Limited, Toronto, Ont. This sand was dredged from a deposit in Lake Ontario, and was free from clay or other deleterious matter. Its chemical composition was as follows:

Loss on ignition, per cent.	16.51
Silica, per cent.	42.81
Iron and aluminum oxides, per cent.	11.76
Calcium oxide, per cent.	24.47
Magnesia, per cent.	1.68

The specific gravity of this sand taken at 70° F. was 2.689.

The original sand was thoroughly dried upon a metal heater and, to remove the larger material, was passed through a sieve having 4 meshes per linear inch. All material passing this sieve was then divided into the portions retained upon a standard set of 8-in. diameter hand sieves having 8, 10, 20, 30, 40, 50, 80 and 100 openings per linear inch. The portions secured were placed in bags and carefully labelled for identification.

The sands used in Concrete tests were not sub-divided as above described, and, in consequence, they contained small percentages of "dust" passing a No. 100 sieve.

Stone.—The crushed stone used in the concrete tests was a composite lot secured from several quarries in Ontario. It was, in general, a dark colored limestone of good quality. Prior to use, this material was divided into the portions retained upon 1, ¾, ½ and ¼-in.

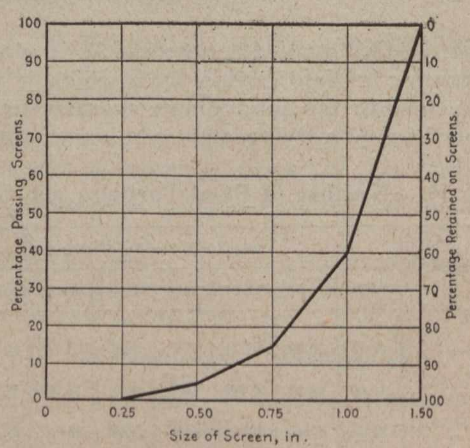


Fig. 4—Grading of Broken Stone

screens. No material was larger than that retained upon a 1-in. screen. The grading of the stone as used is shown graphically in Fig. 4. Its weight was 87 lbs. per cubic foot.

(Continued on page 20)

CORE CONSTRUCTION*

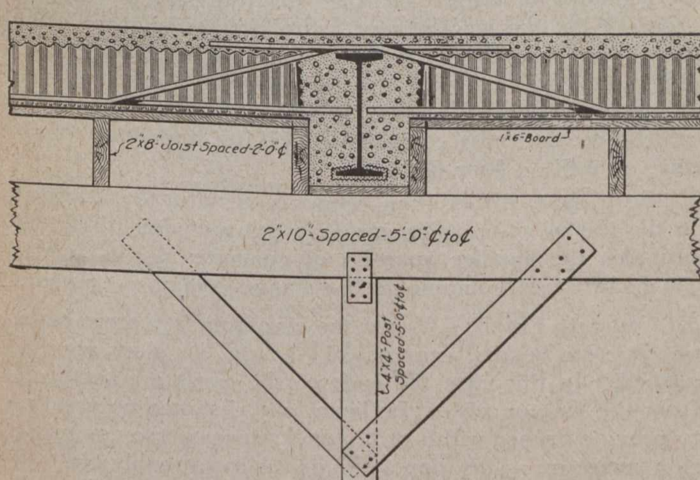
By A. H. Bromley, Jr.
Chief Engineer, Sales Dept., Berger Manufacturing Co.

IN the early days of the development of reinforced concrete, it was realized that the rectangular beam, with a width from top to bottom sufficient to develop its full compressive strength, was not an economical section, therefore attention has been centered on the study of the possibilities of the use of a portion of the slab at either side of the beam as constituting the compressive flange of the beam, and with the stem designed to care only for the protection of the steel and the secondary stresses to which it is subjected.

The first designs of this nature were carried out in connection with the use of slabs which were of such length of span as to require their being reinforced to carry the load between beams. A further development was the reducing of the width of the slab to a point where no reinforcement was required in it, thus effecting greater economy in the construction, as the total steel required in the reinforcement of the beams is not increased by their closer spacing.

As in most cases, the attempt to develop the idea was carried to extremes when considered from a practical and economical standpoint. Among these extreme ideas was included a system of cross-tee beam construction, wherein the depth of the beam stem was reduced by reason of the load being divided up and carried by two beams at right angles to each other. This system also involved bearings being provided on all sides of the floor panel, and form work, the cost of which was greater than the benefit derived.

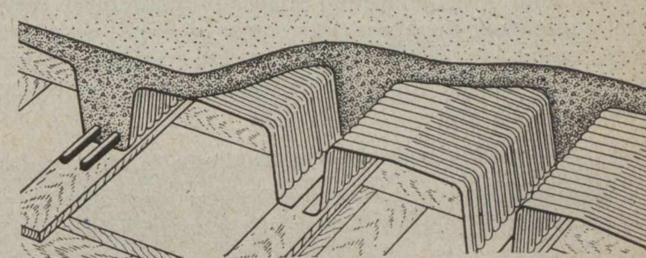
The development of the one-way tee beam from a commercial standpoint, however, also proved very difficult owing to troublesome and expensive form work involved. The first forms were made of wood, necessarily removable, thereby involving smoothed surfaces and rounded corners, which were very expensive, both in first cost and for re-use.



Longitudinal Section, Showing Cores and Reinforcing Rods in Joists

Another step in the development of a suitable form to reduce the cost of centering was the introduction of terra cotta, clay tile or plaster of paris blocks as fillers. These effected considerable saving over the wood form construction as the blocks permit the use of flat wood centering. This method, however, does not give maximum economy, for the block is dead and useless weight that must be supported by the beams, columns and footings, and the freight and haulage charges often make the cost prohibitive.

The core type of reinforced concrete floor construction, in which pressed steel cores are used to produce a series of parallel connected tee beams, has come into extensive use and should be recognized as a standard form of construction. This type of construction is exploited principally by companies manufacturing specialties in pressed steel products and the forms are known by various names, such as "Pressed Steel Cores," "Steel Forms," "Metal Tile," "Floretyles," "Steel Tile," etc., and also known to many as the "tin pan system." This system provides a simple type of floor construction which may be supported by walls or beams of steel or reinforced concrete poured in conjunction with the floor system.



Cross-Section Showing Tee-Beam Construction

Description of Core Details

The various features that go to make up the system are as follows:—

Tee Beams.—The floor structure is in reality a series of connected tee beams presenting a flat surface on top. The flange, usually from 2 ins. to 3 ins. thick, forms the compression member of the tee section, and the web or rib of the tee, which encases the reinforcing steel, projects below the slab. These ribs are from 4 ins. to 6½ ins. in width, generally spaced from 24 ins. to 31½ ins. on centres and from 4 ins. to 14 ins. in depth.

Ceilings.—In the great majority of cases a metal lath ceiling is secured to the bottom of the joists, which, upon being plastered, produces a flat ceiling surface. There is, however, a tendency to extensive use of the construction in buildings for factory or storage purposes, which are necessarily designed for heavy loads, and where the flat ceilings are not required. In these cases the cores are removed, and upon removal a concrete ceiling with a surface similar in appearance to a wood joint floor is produced.

Special Features.—The special feature in the core type of construction is in the use of a broad trough-shaped pressed steel core or centre, placed in an inverted position, and supported by wooden skeleton centering. These pressed steel cores form the centering for the sides of the rib and the bottom of the flange, while the wooden supports form the centering for the bottom of the rib of the tee beam. The cores are varied in design and weight to meet requirements of either permanent or removable centering. Removable cores are usually made in from 16 to 22 gauge and non-removable in from 24 to 28 gauge material. Where necessary, the cores are tapered in width to increase the width of rib adjacent to supports, to make proper provision for the maximum shearing stresses, or for requirements of negative bending moment. Suitable end caps are provided to close up the ends of the lines of cores.

*Paper read at Convention of American Concrete Institute, June 29th, 1918.

Dimensions.—The dimensions of the cores are approximately as follows: Stock lengths, 2 ft. 6 ins. to 4 ft. Regular widths, 20 ins. and 25 ins., with special widths of 6 ins., 8 ins. or 12 ins. and over, made for narrow spaces. Depths, 4 ins., 6 ins., 8 ins., 10 ins., 12 ins. and 14 ins.

Shape of Cores.—The cores are made with sides slightly tapered and usually with a slight arch in the tops.

Beam Spacing.—It may be readily seen from the data given above, that the skeleton wood centering to support the cores and form the bottom of the rib will require horizontal supporting members only 6 ins. or 8 ins. wide and spaced 24 ins. to 31½ ins. on centres, insuring great economy in material and labor as well as speed in erection of the carpenter work.

Methods of Construction

There are two general methods of construction, differing principally in the installation of the metal lath ceiling under the cores.

One method is where a light gauge ribbed expanded metal lath, or a stiff heavy gauge lath is laid over the forms before the cores are placed. The concrete when poured engages the meshes of the lath in the bottom of the ribs, thereby securing it in position. It is generally found advisable, however, to secure wire to the lath, which wire is placed so as to extend up into the concrete rib, and thereby form a more secure fastening.

The other method provides for regular ceiling lath to be installed after the wood centering has been removed from under the construction. The cores are secured by lightly nailing them to the wooden forms, when wire hangers (consisting of a hooked end and straight portion about 15 ins. to 18 ins. long) are pushed through the holes previously punched through the tops or sides of the cores, and when concrete is poured, these hook ends are securely held by it. After the concrete is poured and the centering removed, furring bars or rods are suspended under and at right angles to the ribs by securing them with these wires. The metal lath is then wired to the furring rods in the usual manner. Advantages are claimed for this method in that the ceiling plaster is kept away from the concrete, thus allowing an air space above the entire plastered ceiling. This eliminates discoloration of the white coat, which discoloration is usually in evidence where any type of joist construction is used which does not provide continuous air space. A further advantage of this method is that the centering can be removed for re-use.

Advantages of Core Construction

Among the decided advantages for this pressed steel core floor construction are the following:—

(1) 45% to 60% saving in bulk of concrete over that required in slab construction of same depth.

(2) Saving in reinforcing steel in the floor by reason of light weight of concrete and core.

(3) Saving effected in entire structure, including beams, girders, columns, and foundations of the building by reason of small dead weight of floors.

(4) Weight of cores only one-thirtieth as much as terra cotta tile fillers for equal covering area.

(5) Bulk of cores only one-fortieth as much as tile terra cotta fillers.

(6) One core covers four times as much area of floor as one terra cotta tile.

(7) The forms produce tight centering and eliminate waste of concrete.

(8) Small masses of concrete are used, thus reducing the immense quantities of moisture incident to larger masses of concrete, the drying out of which interferes with the heating of buildings during the first winter season.

Straight and fixed alignment of joints obtained by slightly nailing the pressed steel core in position, and which alignment is necessary to develop the strength of the structure. Reid, in his book on "Concrete and Reinforced Concrete" (1907 edition) page 525 (referring to hollow tile fillers), says: "In the construction of slabs of this type some positive means should be employed to hold the tile rigidly in position until the concrete has been put in and set. The author has seen tile displaced as much as 2 ins. or 3 ins. during progress of construction. The strength of a floor rib would appear to be an uncertain quantity if even a single tile is displaced to any extent."

The danger of extensive damage by failure from defective material or workmanship, is remote, as the floor consists of small units. Therefore, no large areas or excessive loads are dependent upon any one unit.

Details of Design

Refinement.—An inspection of the details of design shows it to be ideal from an engineering standpoint, as all parts can be properly proportioned to stress and fireproofing requirements. This refinement of design eliminates the excess material used, and the unnecessary weight imposed on the entire structure by other types of construction. The main structural parts of the building can also be fireproofed with the greatest ease in connection with the installation of the concrete work in the floors. The system provides for the design of concrete construction with a minimum quantity of concrete, which adequately cares for the shearing and bond stresses, as well as protection from fire to the reinforcing steel.

Depth.—The depth of the construction is determined by economical design of the concrete and reinforcing steel, or by the requirement as to ceiling heights. The flanges are usually designed as 2 ins., 2½ ins. or 3 ins. thick, making the total thickness of the floor structure range from 6 ins. to 17 ins., to which, of course, must be added the thickness of plastered ceiling and floor fill, if any, as well as floor finish.

Best Type.—Maximum economy is naturally attained in the use of 25-in. wide cores, it being evident that this provides the smaller quantity of concrete in ribs, while the quantity of concrete in the flange remains constant.

Concrete Frame.—When the core floor structure rests on concrete beams, economical design is also accomplished. In this case the ends of the cores are set back from the web or rib of the beam any distance necessary, to provide proper width of flange to develop the compression member. This upper flange will probably be of ample depth in that it includes the depth of both rib and the flange over the core.

Application of Formulæ.—While the regular principles of design of reinforced concrete, of course, apply to this system of construction, owing to the thin and narrow slab, and departure from straight lines in outline, the application of the usual tee beam formulæ does not do justice to the section. One way to design quickly and within safe lines will be to work up a table of maximum resisting moments, based on the upper concrete compression flange value. These calculations can be readily made by applying the common theory of beams, which assumes that the stress in compression or tension is proportionate to the distance from the neutral axis. From this moment we can ascertain the greatest amount of reinforcing steel which can be used, without over-stressing the concrete for various depths and widths of cores. Having thus established the maximum value of the tee section we can proceed to design this type of construction by the usual rectangular beam formulæ, with but the one restriction, *i.e.*, that we must not exceed in steel requirement the maximum allowable area previously determined. General rules by empirical formula as to proportion of stem to flange should not be construed to rule in this type of construction, provided we keep within the limits above outlined.

Published Tables.—The various companies exploiting these pressed steel cores have published tables which may be obtained upon request. Undoubtedly, these tables will be revised if standards are prepared by an authoritative body, which standards can be used generally for design and in practice. This will establish with building departments and commissioners a proper rating and basis for recognition of the system.

Reasons for Investigation and Standards

Many architects, engineers and companies supplying reinforcing materials, work by empirical formulæ or "rule of thumb" methods, without considering or realizing the great danger in such practice. There is also the tendency for some designers, when in close competition, and when established regulations are not in evidence, to depart somewhat from good practice.

On the other hand, architects, engineers, building commissioners and others who are not familiar with the merits of the construction or its details, are apt to place restrictions which will bar its use in many places for which the construction is peculiarly fitted.

Believing that he voices the desires of the manufacturers in requesting standards and stating that there is need for the establishment of standards, the author suggests that the American Concrete Institute prepare and recommend standard specifications. Therefore, this paper should not more than mention some of the questions which arise as to details of practice, and regulations with which manufacturers and users of this type of steel centering are confronted.

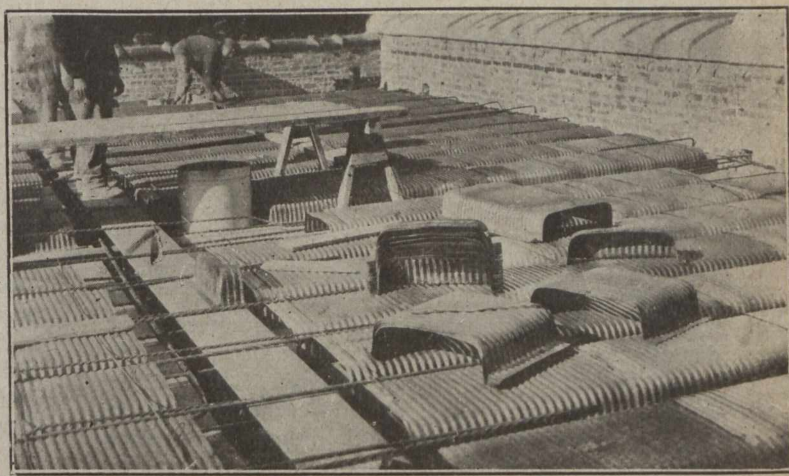
Severe Code Restrictions

Flange Thickness.—In exploiting the sale of cores, we are confronted with codes in which slab thicknesses up to and including 4 ins. are required. It does not appear right that this type of construction should be saddled with restrictions as to the thickness of the slab owing to the possibility of the use of materials which would be dangerous to any type of reinforced concrete construction. It is a well-established fact that "lean mixtures and large size stone" will produce poor results in beams when rods are spaced as per usual standard, and in slabs where reinforcing steel is placed not over 1 in. from the bottom of the slab, therefore, regulations permitting a 2-in. slab based upon the use of 1 to 6 concrete with small gravel or broken stone, should be termed as good practice, where cores and short slabs are used. The surface of this slab is unlike the bottom of a beam, as it has no corners nor sides which can be attacked by flame, yet 1½-in. or 2-in. concrete protection to steel in the bottom of beams is allowed by the majority of regulations.

Flange Width.—We are at times confronted with regulations allowing only three-fourths of the slab width between joists as a compression member. Since there is practically no slab stress there will be no simultaneous action as floor slab and joist flange in the plate of this type of construction, therefore, is it not reasonable to treat the entire width between joists as a compression member?

Steel Protection.—Shall the construction be considered a slab with core, and the protection under and at the sides of the reinforced steel in the joists be treated as in a slab with about 1 in. concrete protection, or shall the joists be considered as beams and the steel be protected by 1½ ins. of concrete? Also, what allowance, if any, can be made in this protection when metal lath and plaster ceiling is installed under the joists? And also, what further allowance can be made if an air space exists above the ceiling?

Determination of Bending Moments.—The author believes that bending moments can be figured as in beam construction, *i.e.*, simple spans ⅛ W.L., semi-continuous spans 1/10 W.L. and continuous spans 1/12 W.L.



Placing Cores and Reinforcing Rods

General Use and Installations

This floor system is at its best when used for spans up to about 32 ft. in buildings designed for light floor loads, and when flat ceilings are desired, such as in residences, hotels, apartment houses, club buildings, retail stores, schools, hospitals, theatres, etc.

Among prominent installations are the Illinois Central Office Building, Chicago, a nine-story building in which a total of over 33,000 lineal feet of 22-gauge removable cores were used, nearly all of which were used three times to complete the job. About 11,000 28-gauge end caps were used.

The Engineering News-Record of November 29th, 1917, describes the use of these cores in the balcony construction of the Vanderbilt Theatre, W. 48th Street, New York City. We also note recently in engineering papers an account of the use of these cores in one hundred workmen's houses built of concrete at Donora, Pa.

Among other installations are the La Salle and Koch Department Store, Toledo, O.; White Plains Court House, White Plains, N.Y.; Austin High School, Austin, Tex.; Huntington High School, Huntington, W. Va.; Book Building, Detroit, Mich.; First Regiment Armory, Chicago, Ill.; St. Paul's Cathedral High School, Pittsburg, Pa.; Painter-Dunn Co. Garage, Pittsburg, Pa.; May Co. Department Store, Cleveland, O.

Large quantities of these pressed steel cores have been sold and no one can give an accurate estimate of the number of square feet of floors and roofs which have been constructed with this system.

DISTINGUISHING MANILA FROM ALL OTHER "HARD" ROPE FIBRES*

By Charles E. Swett

IN the rope industry fibres are classed as "soft," hemp, jute, linen, cotton, etc., and "hard," as manila (musa), sisal (agave), maguey, New Zealand (*Phormium tenax*) and others. In general, the hard fibres are lignified. Among the soft fibres jute also is a lignified fibre, but its physical characteristics place it beside hemp.

The two hard fibres of pre-eminent importance are manila and sisal. The distinction between manila and sisal is not easy, except by a practical ropemaker, and not always by him, especially when they occur together in rope.

When it is desired to estimate the amounts of the two present in a given rope the microscope has to be used. The cross-sections are characteristic but not in all cases satisfactory. For instance, sisal from East Africa is sufficiently different from manila to enable one to separate them, but sisal from Yucatan (henequin) is not always enough different to make it possible to say how much may be present.

The sections are not easy to make and when made may contain thousands of fibres. When it is considered that the field under magnification to 200 diameters is of the order of one millimeter and that a rope of 2 or 3 ins. of cross-section may have to be examined, it will be understood that some method for differentiating these fibres other than the employment of the microscope would be of great use. Such a method has been worked out in the laboratory of Arthur D. Little, Inc., and is here described in some detail.

If the sample is treated with a solution of bleaching powder acidulated with acetic acid, then with ammonia, manila takes a russet-brown color. All other hard fibres turn cherry-red. Thus it becomes possible to distinguish manila from all the others, which is the matter of chief importance.

Solutions Required

1. Ether, to pour down a strand to remove most of the spinning oil.
2. Bleaching Powder Solution.—A clear solution of chloride of lime, containing about 5 per cent. of available chlorine, acidulated with acetic acid (30 c.c. of bleaching solution and 2 c.c. glacial acetic acid).

Acidulation with an acid stronger than acetic will not answer; for example, hydrochloric acid will give no test.

3. Water to rinse after the above.
4. Alcohol to remove water.
5. Strong Ammonia.

Preliminary Examination of the Sample

Remove most of the oil by pouring ether down the strand. Wave through the air for a minute or two to remove most of the ether; immerse one end of the sample in the acidulated bleach solution for 20 seconds; rinse first with water, then with alcohol and then immerse in ammonia.

Manila will instantly turn brown.

Sisal, New Zealand, istle, Mauritius, maguey will assume a cherry-red.

When so applied the test is somewhat fugitive, the red color degrading in the course of a few minutes so that it may not be possible to pick out the different colored fibres from the strand. As applied, it enables one to say whether the sample is all manila, all non-manila or a mixture. This is all that is required in many instances.

When it becomes necessary to estimate the percentages of manila and non-manila the procedure is as follows: Apply the test as before but instead of immersing the fibres in ammonia in the last operation, suspend the treated end of the strand above the ammonia for a minute or so. As thus practiced the manila does not assume the brown color as rapidly, but at the end of 2 or 3 minutes the color develops and is permanent. The cherry-red of the non-manila fibres remains for hours and a separation may be made by picking out the red or the brown. A reading glass is of assistance.

When the separation is made as above it is desirable to take the two differently colored strands and apply the test as first described, that is, by treating the hitherto untreated ends with ether, immersing in bleach acetic solution, rinsing with water and then immersing in ammonia. This serves as a check and as the separation will be closely approximate as a result of the fuming test, the few fibres which may show up as wrongly placed can be removed with ease.

One desiring to practice this test should first work on samples of known origin. With practice it seems to be possible to estimate the manila content of a rope down to a single fibre. As the test is so quickly applied it is the writer's practice to take less than a strand for treatment and then go through the sample taking, perhaps, 20 or 30 fibres at a time.

The difference between the red and the brown is most evident at the end of 3 or 4 minutes after fuming with the ammonia.

The bleach solution made with one part of chloride of lime and seven parts of water, then filtered, may be kept in a stoppered amber bottle away from the light for a long time. When some is to be used it should be poured from the stock solution and acidulated with the acetic acid for present use. Throw away when the tests are done; it will not keep in an acidulated condition. Chlorine water will not serve, neither will iodine solution or bromine water; nor will any acid tried answer so well as acetic.

Too much emphasis cannot be placed on the fact that hydrochloric or other strong acid is not suitable to replace acetic acid as used in this test. This reiteration is made because, notwithstanding specific instructions, two competent chemists have assumed that because the test failed with hydrochloric acid there was nothing in it. In neither case were the directions followed and when attention was called to this fact no trouble was experienced in securing the appearances described.

*Report of sub-committee presented at Convention of American Society for Testing Materials, June 29th, 1918.

An application is pending before the Department of Communications and Public Works, Mexico, for the construction of an electric railway connecting Tampico with Mexico City, passing through the capitals of the States of Vera Cruz, Tlaxcala and Puebla, as well as many other important towns.

Though Switzerland led the world in the early electrolytic production of aluminium, France has been the chief producing nation since 1896, and supplied as much as 39.2 per cent. of the world's annual total at the time of the outbreak of the war. The ten plants for electrolytic aluminium had in 1914 an aggregate of 140,000 h.p., with a capacity of 19,000 tons a year. The three plants of the United States supplied 25.8 per cent. of the world's production; the two plants of Switzerland, 12.4; the two English plants, 7.8; the one Canadian plant, 5.2; and the two plants of Norway, 4.3. French supremacy is due to the superior quality of the bauxite mined.

LOCATION OF EARTH ROADS*

By R. W. E. Loucks

THE question of good roads is one that has been before the people of Saskatchewan from the time of the arrival of the first group of settlers more than half a century ago, but it is only within the last few years that we are beginning to realize its importance. A fresh impulse appears to be spreading not only throughout the province, but clear across Canada and in all the civilized nations of the earth, having for its object the improvement of our highways. The success of the good roads movement depends on educating the public to its many great advantages and convincing them that every dollar wisely and efficiently spent for this purpose is an investment that will pay an interest that cannot be computed in dollars and cents.

The "back-to-the-land" movement will never be entirely successful until the road question has been more thoroughly considered and means of communication between the farm and the town provided for settlers who are being asked to take up farms in the various parts of the province. The social life on the farm is one of the causes of farmers' sons and daughters leaving the farms and drifting to the city. Good roads will provide convenient means of communication between scattered farmers and thus remove much of the monotony of farm life. Much can be done to reduce the high cost of living with which we are faced by the improvement of our roads. By reducing the cost of transportation we are directly or indirectly reducing the cost of living. Before war prices prevailed for farm products, most if not all the profits of producing grain in certain districts was eaten up by the heavy cost of marketing such produce over the long and bad roads.

In a province such as Saskatchewan where the supply of gravel for surfacing roads is somewhat limited and as yet undeveloped and where distances are so great and settlement so sparse, a large percentage of our public highways must of necessity be earth roads.

Planning of Roads

Much can be accomplished in road making by using the proper materials and equipment, by efficient organization and supervision of construction, yet this may be more than offset by bad planning. Bad planning of roads retards the progress of any community—by increasing the cost of haulage up steep grades and over greater length than is necessary. This tends to scatter the population so that the amount of road to be built and maintained is not in keeping with the financial status of the taxpayer.

Roads should be located by engineers governed by the principle of balancing the lengths and grades that a minimum effort will be required to go from one end to the other. Since the introduction of self-propelled vehicles, with their greatly increased radius of action, the loss of time, energy and money caused by lack of directness in highways has been more apparent, and it is obvious that any intelligent plan for the improvement and extension of a new system must include the correction of alignment in existing roads and provision for direct routes between important centres.

In Saskatchewan the engineer is given very little latitude as to the location of roads. He must adjust his scheme to fit into the system of rectangular sub-division into which this province is surveyed. This is not always an easy task. In passing, however, it is doubtful if any better system of laying out our prairie lands, having regard to all the conditions, could have been evolved. On the other hand, the sooner we abandon the idea of sticking to these rectangular allowances, whatever obstacles may be encountered, and freely divert our roads to suit the topography of the country, the sooner will we lay the foundations for a wise and permanent system of highways.

In locating our roads we must remember too that generally speaking, the road will be constructed from the material over which it passes, so it will be well to study the nature of the soils and avoid where possible swamps, sloughs, alkaline areas, etc. This is desirable not only on account of the poor nature of the road-making qualities of the soil, but also on account of the difficulty of draining.

In any system of highways there must be certain focal points of varying importance. Traffic increases as such points are approached, and hence the capacity of the roads must increase in like proportion. The gradients of main thoroughfares should be made as light as possible, both for the reason that loads are heavier, and that lighter gradients permit higher speed and therefore greater capacity. Present centres will increase in importance, but not in the same ratio. What is to-day a subordinate centre may in another decade be of relatively greater importance, and the highways approaching it may have to be widened and improved in alignment; hence it is important to give careful consideration to the location and width of each particular road of the system in order that the cost of such future improvement may be minimized.

The principles which should govern the creation of a system of highways in a city are quite similar to those applying to a state or a country. The city will also have one or more principal traffic centres and a number of subordinate centres. Direct connections between these centres are of prime importance owing to the heavier loads and the lower speeds enforced by greater volume of traffic. It is a mistake to treat a city and a contiguous rural system of highways independently of each other. The motor car has placed the city in such close relation with the rural districts surrounding it and with the neighboring towns, and their interdependence has become so great, that the entire system of highways, both rural and urban, must be considered as an entity.

The chief highway connecting two towns should afford a direct connection between the important thoroughfares of each. Frequently this is not the case, but the main and sometimes the only adequate road connecting them leads at either end into narrow, tortuous, and shabby streets which must be traversed in order to reach the business and administrative centre. How different an impression one will obtain and how largely will the pleasure, comfort and convenience of a journey be increased if the approach to a town is through a dignified, well-paved thoroughfare constantly increasing in importance and interest until the climax is reached at a well-designed and convenient civic centre.

It will frequently be necessary to create new and widen existing thoroughfares. As the village grows into a city and the city into a metropolis, such improvements are sometimes imperative. Failure to provide them will prevent or retard the growth of the town, but their cost will often be so great that they will be undertaken with hesitancy. What is commonly called "city planning" consists in a large measure in the correction of blunders made in the original system of highways. Failure on the part of the engineer to exercise some foresight and even imagination in the design of a rational and adequate system of roads is responsible for the incalculable loss caused by an unsuitable street system and the enormous

*From "Western Municipal News."

cost of changing it into a proper one in a densely populated city.

A city does not and should not grow by accretions of fully developed areas; neither should its street plan expand by the addition of one completely plotted area after another. Its growth is not like that of the human frame, beginning with a structure complete in every detail and gradually increasing in size. The skeleton should be created first. The position of the various parts of this skeleton will be controlled by the topography and by the crude but suggestive system of roads which has already developed, each of them for a fairly good reason, namely, that it leads where people wish to go. Others who come to these now unimportant centres will doubtless wish to go in the same direction, and these roads will naturally become the important streets of this new part of the city. Additional connections will naturally suggest themselves, and the entire street system will, if intelligently controlled, be developed in a rational manner along the lines of least resistance, and this part of the city at least will be adapted to convenient movement, orderly business and wholesome living.

In the creation of radial roads a difficult problem is frequently presented by the same town or village through which such roads would naturally pass. It rarely happens than any of the town or village streets are of sufficient width to provide the needed capacity. Their widening would involve expense which might be prohibitive and would be violently opposed by those who, attracted by the quiet of the village street, have made their home there and do not wish to see it converted into a busy thoroughfare. In such cases it is usually possible, by a slight detour, to avoid the local street, save the expense of widening and return to the existing road at a convenient point beyond the town. At the same time, new impetus will be given to the development and improvement of the land along the new highway, while the character of the older portion of the town will be preserved. This plan of adopting the "by-pass roads" is simply following the lines of least resistance and may go far towards overcoming the local opposition which frequently develops when any change is suggested.

Drainage

Drainage is the most important factor in the construction of roads and we cannot get proper drainage if the road has not been properly located. In rolling country or open soil the drainage may not be a difficult question, but in a level country with heavy soil such as we find in the district immediately surrounding our own city, it often becomes a most difficult problem. Here skilful engineering is required to obtain proper grades for the drainage ditches and to locate offtakes which will carry the water away from the road ditches to natural water courses so that the road may dry quickly after rains or in the spring.

A road can never be kept hard and smooth if the water from the surrounding land drains on it. This can often be avoided by taking a little longer route or nearby ground at perhaps a little greater initial cost, but certainly at a much reduced expense for maintenance. Water is, without doubt, the most destructive agent to any class of road, but much more so to an earth road, which is more susceptible to the action of water, more easily dissolved and moved by it, than any surfaced road; for this reason too much attention cannot be given to this problem. This is true in all climates but more particularly in a province such as ours, where rains are prevalent during certain seasons of the year and where the winter months are attended with severe frosts and snowfalls.

The question of grade is also one requiring careful consideration. Not only grade reduction on steep hills, but also the grade through rolling and through level country. In level country the road will be constructed of material from the ditch. Very rarely is a country so level that the quantity of earth taken out of a ditch is the same at every point in the ditch. Some places there will be more earth than is required for the building of the road at that point and at other places not sufficient. Careful thought must be given to the disposal of this earth, whether it should be wasted and earth borrowed at the light cuts. In rolling country where ditching is necessary, the grade must be so planned that the cut from the hills must just balance the fills up to an economical length of haul, which length of haul must be determined.

In grade reduction on hills the problem is to establish the most economical grade. This grade will vary with the relation of the cost of the grade reduction to the amount and nature of the traffic. It will be dependent on the direction of heavy traffic, the type of road, or perhaps be governed by a ruling grade on some other portion of the road.

It has been calculated that a horse of, say, 1,200 pounds, will by exerting a force equal to one-tenth of his weight, draw a load of 2,000 pounds on a level road; on the same road but on a 5 per cent. grade, with the same force against his collar, he will draw 1,000 pounds, and on a 7 per cent. grade only 750 pounds. What is true in connection with horse-drawn traffic is at least equally true in the case of the mechanically drawn traffic.

Alignment

A proper alignment of the road is necessary both for appearance and economy. In level country a well-aligned road grade and ditch has an appearance in keeping with a neat farm yard and farm buildings. On side-hill work a change in alignment may make a considerable change in the grade of the road or in the cost of construction. In rolling country it is often found more economical to build a road around a hill than to build over the hill and thus reduce a heavy grade to a level or nearly level grade.

It is contended by those who advocate the construction of relatively permanent grades for country roads that such grades will have a considerable value as foundations for pavements after the expiration of their life as a wearing surface. This is undoubtedly true if the alignment and grades on the highway are satisfactory at that time. Otherwise the old grade will constitute a liability rather than an asset. May not, then, the added cost of right-of-way grading necessary to make the location the best obtainable be offset to some degree by the value of the old grade at the expiration of its life?

Straight alignments offer certain advantages; they represent the shortest distance between two points, and offer least obstruction to all kinds of traffic, but their length is usually limited, especially in hilly districts, owing to the numerous obstacles to be met with. Roads should also be free as far as possible from sudden deflections, which should only be permitted where there is good reason for abrupt change in direction.

Curves

The utility of curved roads in special circumstances cannot be ignored, and they are almost a physical necessity in some cases to suit the contour of the ground. Further, curved roads have the advantage of changing the direction of traffic gradually instead of abruptly, and they are frequently necessary for the avoidance of obstacles or the preservation of features of interest, whilst their aesthetic value is undeniable.

Whilst I can foresee objections to the use of curves on road location by the surveyor on account of intricacies of location and definition on the ground or on account of involved mathematical calculations, and further objections by the superintendent of construction who in the majority of cases knows nothing whatever concerning the theory of curves or the manner of reproducing same on the ground, from the extreme posted limits, yet I think these objections can be more than counterbalanced by the added attractiveness of a graceful curve, and by the fact that the constructed grade instead of swinging from one surveyed limit of the roadway to the other will more nearly follow the centre of the right-of-way. The customary method of making a road diversion at the present time, except in cases where the road adjoins a curved railway right-of-way, is to locate a number of points on one or the other of the limits of the roadway and join such points by straight lines. When the road is constructed, more than likely the grade takes a winding course rather than parallel to the surveyed limits of the roadway. This brings the grade close to one of the surveyed limits, and when the improvements are made such as widening of the grade, the result quite frequently will be either hampering of the work or an encroachment on the adjoining lands. The more extensive employment of curves in locating such roads will remove most, if not all, the objectionable features in our present surveys and will at the same time provide an alignment in keeping with the needs of the present-day traffic.

The curves should be planned to afford a sight distance; that is, the greatest distance at which the drivers of two approaching vehicles may see each other's machine, of not less than 250 feet. This would afford the driver of each of the approaching vehicles a distance of 125 feet within which to come to a halt. In the case of automobiles travelling at the rate of 15 miles per hour, a maximum of 6 seconds would be provided to bring either machine to a standstill before collision.

The radii of the curves will depend on the speed of the traffic. On the level stretches of roads and on those with gentle gradients, upon which naturally motors are driven at top speed, sharp curves are especially troublesome and dangerous at the same time, if they interfere with the proper outlook. In hilly districts, on the other hand, motor cars going both up and down hill are naturally obliged to proceed at much lower speeds, at which they can easily take the sharp curves which have to be adopted on such roads owing to the limitations imposed by the configuration of the ground.

It would seem advisable to set a minimum radius of 300 feet wherever possible without incurring a prohibitive cost. If the radius has to be shortened to 150 feet and under, danger signs should be placed at least 400 feet from the B.C. and the E.C.

When a curve occurs on an ascent, the grade at that place should be diminished in order to compensate for the additional resistance of the curve. When it is necessary to make a radius of curvature less than 300 feet, we should follow the principle of reducing the grade on the curve at the rate of 1 per cent. for every 50 feet that the radius has to be reduced; so that where we are locating a 5 per cent. grade and have to put in a curve with 200 feet radius, we give a 3 per cent. grade. On curves with a small radius the employment of transition curves is desirable. As a rule, there ought to be a tangent of about 100 feet between two curves. Quick reverse curves are disagreeable and dangerous with automobile traffic and are to be avoided.

The curved road around the hill is often no longer than the straight road over it. In addition, a more or less winding course is an advantage from a maintenance standpoint, as on the winding road the wheel traffic has a tendency to spread over the entire surface.

Where the road is to be located along the slopes of ridges and a choice of ridges is available, those affording the greatest protection against the action of prevailing storms should be selected. The advantages resulting from such a choice will be greater freedom from the effects of heavy rains and drifting snow.

Street Intersections

A development in street improvement resulting from automobile traffic is the demand for curves of larger radius at the corner of street intersections. Past practice resulted in a corner that was nearly square, or had the curve varying from 4 to 10 feet radius. This means that the vehicle could not turn into an intersection street until it had passed the corner. The driver of a car must either shift gear and slow down to a suitable speed for the turn, or else he may take the turn with an abruptness that is a menace to other vehicles as well as pedestrians. A further alternative practised by many drivers is to swing widely at the turns, a practice which carries them to the wrong side of the roadway and risks accident.

Toronto, Ottawa, Windsor, Stratford, Chatham and other cities in Ontario are widening and rounding many of their street intersections. The increasing radius of the curves enables an automobile to follow a course parallel to the curb and keep to the right side of the roadway, with greater convenience and safety.

Systematic Planning of Roads Prior to Construction

For the best results in obtaining good roads, the whole road scheme should be planned in advance and every piece of work done should be a step towards the completion of the finished road. This systematic construction is very important where drainage is necessary. The only proper place to start a road drain is at its outlet, and the work carried towards its upper end. The mile of road which is most used is the one next to the market. It is, therefore, reasonable that the roads nearest the market should be built up first and the system added to each year as far as possible.

In the carrying out of such a scheme the roads of the country should be divided into three classes, *viz.*: the Dominion, provincial and municipal highways. With the first class of road, only the Dominion Government should be concerned, and they should consist of the leading highways east and west and north and south. The work of location, construction and maintenance should be placed under the control of the Dominion engineers. The second class of road, which would be known as the provincial highways, should consist of the roads which, while not being classed as inter-provincial roads, would yet be of direct benefit to the entire province, and should be constructed and maintained under the supervision of the provincial department of highways. The third class of road would be known as the municipal road and would consist of the market roads running between and connecting the main travel routes. These should be the only ones placed under the control of the municipal councils.

Every scheme should be planned in advance by a qualified engineer or other experienced person capable of undertaking such work. The scheme should be far-reaching and so far as possible permanent in character. All records such as plans, profiles and field notes should be neatly labelled and filed with the secretary of the municipality or other central authority, where they would at all times be available to anyone connected with the road scheme. In cases where the engineer from whatever cause might be forced to leave the locality which he has been supervising, his successor would be enabled to step in and carry on the work intelligently and efficiently. With this system in force a great deal of the wasted effort and expense would be eliminated and the entire Dominion would benefit from the development which would follow the adoption of a policy of this nature.

Summary

The most essential considerations which should govern the location of the individual road or a complete highway system may be briefly summarized as follows:

1. Adequate drainage must be secured by systematic planning of the complete road system and avoidance where possible of low areas and objectionable soils.

(2) The grade must be so planned that a minimum of effort will be required to go from one end of the road to the other and so that a minimum of waste results from construction, having regard to ruling grades on the particular road in question.

(3) The road while not monotonously straight should be as direct as the topography and limiting grades will permit.

4. Danger points should be avoided at any cost. "Safety first" is a good slogan for engineers on road location to adopt.

5. Roads should be made interesting where possible. Picturesque vistas and extensive outlooks are desirable as well as easy gradients and a smooth surface.

6. Every road should form a part of a scheme, which scheme should be planned in advance of location and construction under the guidance of a competent engineer.

THE RAIL SITUATION*

OWING to war conditions, the amount of rail purchased by the railroads during the past year has been much less than usual and consequently the tonnage rolled has been less. The routine work of testing at the mills has been carried out as usual, but there has of necessity been less experimental work connected with rails.

Records of performance in service of different types and weights of rails have been reported as usual and these reports are serving a useful purpose, but it is becoming more and more apparent that general summaries of these records are not going to be of a great deal of value. The difficulty is in the absence of any record of the ton mileage carried by the rail before failure. The reports of failures would be of much greater value if the comparisons in the general summaries could be made on the basis of tonnage over the rails in addition to failures per year or period of years. The general summaries do, however, show the general trend is toward a reduction in number of failures, but care should be used in making individual comparisons based on these summaries.

Straightening

The severity of the fibre stress produced in the rail in straightening in a gag press is dependent on the length of the portion of the rail over which the stress is distributed at each blow in the gag and also on the amount of inertia of the rail section. Therefore, with the stiffer sections of the recent heavy-weight rails, it is desirable to increase the distance between the supports in the straightening presses. It is probable that the standard distance of 42 ins. now prescribed in the specifications will have to be increased to meet the necessities of the heavy rails toward which the railroads are tending.

Weight of Sections

Some of the railroads that have increased the weight of section, particularly where this is decidedly over 100 lbs. per yard, report greatly reduced cost of track maintenance and reduction in number of failures as well as better riding track. One of these roads now has in contemplation a section weighing 200 lbs. per yard. While this may be extreme and only to be considered by roads having extremely dense traffic and the heaviest wheel loads, the tendency toward heavier sections seems to be general.

Heat-Treated Rails

From time to time experiments have been made in the quenching and annealing of rails, sometimes of standard sections and sometimes of sections slightly modified to better meet the stresses incident to quenching. The rails when put in service generally showed markedly greater resistance to abrasion, but some failed due to brittleness in track. There are now in track, at points where the service is particularly severe, quenched and annealed rails which have not proven brittle, although they carry very heavy traffic under severe service conditions on heavy curves. The results of these tests indicate that the heat-treated rails may have a future.

Distortion of Rails Under Wheel Pressure

Valuable work has been done by a committee of the American Railway Engineering Association in determining the effect of wheel loads of various weights in producing distortion of the rail head. The reported results indicate that marked distortion is produced by weights considerably under those in common use to-day. The relation of this distortion to rail failures has yet to be determined, but the results already obtained throw considerable light on just what is happening to the rails in track.

*Report of sub-committee presented at Convention of American Society for Testing Materials, June 27th, 1918.

REPAIRED CONCRETE WATER TANK AND WELL ROOF WITH CEMENT-GUN

REPAIRS to a concrete water tank were effected at Waltham, Mass., by the use of the cement-gun. The tank did not have a waterproof lining and there always had been more or less seepage, accompanied by disintegration and spalling of the concrete. Repairs were made during the winter of 1915 and 1916 with mortar applied with the cement-gun. Notwithstanding adverse conditions due to very cold weather, the small amount of work done, near the bottom, was successful. It was decided, therefore, last summer that there was good reason to continue this treatment to the whole tank.

All loose and disintegrated material was removed by pneumatic drills and a coating of fresh mortar applied. The work, according to the last annual report of Bertram Brewer, superintendent of the Waltham waterworks, stood up well during the trying winter of 1917 and 1918.

The tile roof at well No. 1 was also considerably disintegrated by the frost. The disintegrated tiles were removed and, after smoothing the surface with a 1 to 1 mortar, which was allowed to set, felt was laid over the whole and a 1-in. reinforced "gunite" roof was built on top by the use of the cement-gun.

LOSS OF HEAD IN CORPORATION COCKS AND SERVICE PIPES*

By B. J. Bleisteine
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SEVERAL years ago a series of tests were made by the Bureau of Investigation and Design of the Department of Water Supply, Gas and Electricity, New York City, to determine the losses of head in corporation or service cocks and service pipes. The results of these tests may prove of interest and value to the waterworks profession, particularly as to the tests made of the corporation cocks, there being very little data available on that subject. All sizes in use by the department at that time were tested. Of the two largest sizes, four of each type were tested, and of the smaller sizes, two of each type. Of the lead and galvanized service pipes, four lengths of each size about fifteen feet long were tested.

Observations Carefully Made

The observations were very carefully made, the losses being measured by mercury deflections, and the discharges by a two-inch meter of the current type. That type was selected rather than a meter of the disc type, as the flow through the latter caused considerably more vibration and fluctuations of the mercury deflections than a meter of the current type. The same meter was used during the entire series. It was rated on a testing machine at the beginning and end of tests, the exact discharge at various rates being determined by weighing. The corporation cocks were inserted in a length of 12-inch cast iron pipe capped and plugged at the ends, by a tapping machine, in the usual manner. The main was connected to a fire hydrant through the plugged end by means of a short piece of 4-inch galvanized iron pipe. The cocks were set about a foot apart, the 2-inch cocks being set nearest the capped end of the pipe in order to eliminate any uncertainties due to swirls at the high rates where the water entered the main. New lead pipe of a size corresponding to the size of the corporation cock and of double A weight was connected to the tail piece with the usual form of wiped joint. The meter with a control valve at the outlet end was connected up in a similar manner. Both the lead and the galvanized iron pipe were of the usual commercial sizes, the galvanized being of standard weight. The lengths were selected at random from a large stock.

Measuring the Losses

In measuring the losses through the corporation cocks, a U-tube partly filled with mercury was used. One leg was connected by means of rubber tubing to a ¼-inch opening in the top of the main, and the other to a 1/16-inch opening in the wall of the lead pipe, one foot from the end of the tail piece. The rubber tubing and the upper half of the U-tube were completely filled with water. The loss measured included the velocity head, entrance head, and the frictional losses through the cock and the one foot of lead pipe. This total loss was high, and at the high rates of flow it was necessary to connect three tubes in series, the loss being the sum of the deflections in each tube. The losses in the pipe were measured in a similar manner, the two legs of the U-tube being connected to 1/16-in. openings, two and four feet apart. "Tee" connections were provided at the different openings and it was thus possible to measure the losses in the corporation cocks and the various sections of each length simultaneously. The sum of the losses in the various sections was used in computing the unit loss for each length.

How a Burr Was Overcome

Some difficulty was experienced at first in preventing the formation of a burr on the inside surface of the lead pipe in drilling the holes through which the pressures were transmitted. This was finally overcome by punching the holes through the walls from the inside in the following manner: Two ⅛-in. holes were drilled partly through the walls of the pipe on points diametrically opposite, and one of them continued as a 1/16-in. through the wall. By inserting a piece of solid drill rod through this opening, the opposite wall was punched through, leaving the inside surface quite smooth. The burr around the inside of the drilled hole was removed as much as possible with a special tool, and the hole plugged up and soldered. After the tests were completed, the pipe was cut open at each of the holes and the inside surface examined. In only two cases was a burr found, and in both it was very slight. A single 1/16-in. hole was drilled through the wall of the galvanized iron pipe at each pressure point, using a high-speed drill. A small burr formed but no attempt was made to remove it, it being felt that this burr would not affect the results materially, as the inside surface was already quite rough.

Three Observers Used

In taking the readings three observers were used, one to read the meter, the others to note the U-tube deflections. At each rate of flow a sufficient quantity of water was allowed to pass so that the dial hand on the meter made a complete revolution. In this way any errors due to eccentricities of the dial hand or inaccuracies in the gradations were eliminated. The time was taken with an ordinary watch, readings being taken to the nearest half-second. As the total time for any individual test was never less than two minutes, the resulting error was probably less than 1 per cent. The deflections were read at fifteen-second intervals, the readings being taken to the nearest 1/16 of an inch. As no deflection was less than 1 inch and at least eight readings for each rate were taken, the error is probably also well within one per cent. After a sufficient number of tests had been made on each length, the unit losses at the observed rates of flow were computed, one inch of mercury deflection being equivalent to 1.047 feet of water. These values were substituted in the Williams and Hazen formula,

$$v = c \quad r^{0.63} \quad s^{0.54} \quad 0.001^{-0.04}$$

and the average value of c for each length obtained. From these the average value of c for each size was computed. The values of c for the same section of pipe at different rates of flow showed very little variation, the extremes varying not more than three per cent. from the average. For different sections of the same length of pipe, however, the variation was considerable, even in the case of the lead pipe. The ½-in. pipe shows the greatest difference. If the same difference had prevailed in the pipe upon which the loss of head tests were made and the actual diameters had been used in the computations, the effect would have been to reduce the values of c somewhat. In the case of the ½-in. this would have meant a reduction of 4 inches. However, as it is usually quite difficult to measure the diameters accurately, values of c computed on the basis of the standard diameters would be of the greater value.

With one exception, the losses through the cocks with 90-degree tail pieces are larger than those with 45-degree

*Abstracted from paper read before the American Waterworks Association.

tail pieces, the difference being greater for the smaller sizes. The addition of a strainer or eel guard practically doubles the loss.

The relation between the loss in the corporation cock and the velocity head may be expressed by the formula $C = H/P$, in which H is the velocity head, and P the total loss as measured by the U-tube minus the loss through the one foot of lead pipe. The value of C is not a constant, being in general higher for the larger values of H. The loss P also represents the pressure required to produce the velocity head H, hence C is equal to the coefficient of discharge. The following are the values of C:

	$V = 10.0$ ft./sec.	$V = 30.0$ ft./sec.
½-inch—		
45 degrees T. P.	0.433	0.467
⅝-inch—		
45 degrees	.496	.500
90 degrees	.367	.500
¾-inch—		
45 degrees	.433	.443
90 degrees	.380	.408
1-inch—		
45 degrees	.445	.445
90 degrees	.478	.478
1½-inch—		
45 degrees	.577	.590
90 degrees	.567	.548
1½-in. strainer—		
45 degrees	.351	.298
90 degrees	.300	.322
2-inch—		
45 degrees	.459	.467
90 degrees	.433	.431
2-in. strainer—		
45 degrees	.306	.326
90 degrees	.292	.308

The relation between the losses in the corporation cocks and the losses in pipe can be shown by computing the length of lead pipe of a size corresponding to the size of the cock, in which the loss would be the same. This equivalent length is not a constant, being greater for the higher rates of flow and for the larger cocks. The above table gives the range in the values of this length for the various sizes and rates of flow indicated.

Size and type.	Range of discharge, gallons per minute.	Corresponding range of equivalent length of lead pipe.
½-inch—		
45 degrees T.P.	3.5— 22.5	3.86— 4.70
⅝-inch—		
45 degrees	6.0— 35.0	4.96— 5.94
90 degrees	5.0— 35.0	6.24— 7.54
¾-inch—		
45 degrees	8.0— 50.0	5.67— 7.17
90 degrees	7.0— 50.0	6.65— 7.72
1-inch—		
45 degrees	14.0—100.0	7.47— 9.91
90 degrees	16.0—100.0	7.00— 9.23
1½-inch—		
45 degrees	70.0—250.0	10.00—12.38
90 degrees	70.0—250.0	10.22—13.52
1½-in. strainer—		
45 degrees	30.0—160.0	14.10—23.45
90 degrees	25.0—180.0	18.20—21.45
2-inch—		
45 degrees	60.0—400.0	22.60—28.50
90 degrees	60.0—400.0	22.95—31.10
2-in. strainer—		
45 degrees	80.0—350.0	34.00—40.00
90 degrees	80.0—300.0	36.00—40.75

CHLORINATION AT DETROIT, MICH.

At Detroit, Mich., the cost of chlorination treatment for the year ending June 30th, 1917, was approximately 40 cents per 1,000,000 gallons, according to the last annual report of the board of water commissioners. The character of both the river and the treated water was remarkably good, the bacterial counts and the positive tests of B. coli both having been considerably below the records of previous years. The only treatment given to the water was by means of chlorination, the quantity of liquid chlorine used averaging about 2 lbs. per 1,000,000 gallons of water treated; equivalent to one-quarter of one part of chlorine per 1,000,000 parts of water. The treated water showed an average of 7 bacteria per c.c. on agar at 37°, and confirmed evidence of B. coli in only 16 out of 2,945 samples of 10 c.c. and no positive indication in any of 589 tests of 1 c.c. samples.

GERMAN SAMPLES WANTED

Manufacturers' agents who wish to obtain Ontario agencies of United Kingdom firms, now or after the war, are invited to correspond with the Department of Overseas Trade, 257 Confederation Life Bldg., Toronto. This department is a branch of the British Board of Trade, and has official status, the head of the department having cabinet rank.

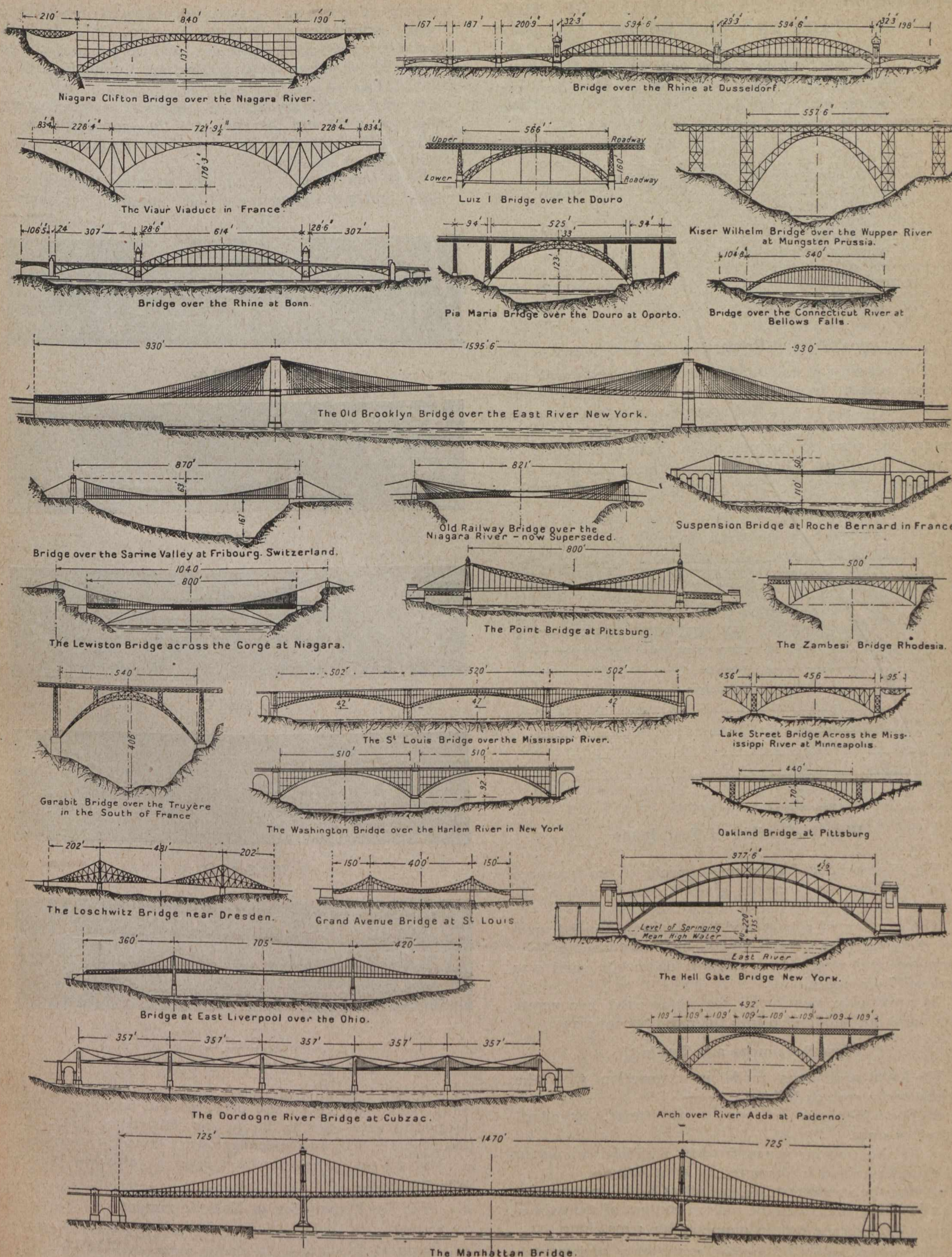
We are informed by Fred. W. Field, who is the trade commissioner in charge of the Toronto office, that he will be pleased to discuss with agents and importers any obstacles to the encouragement of British trade in Ontario aside from shipping difficulties or other troubles purely incidental to the war.

Samples of German or other enemy merchandise sold in Ontario prior to the war will also be useful to the work of the department. "There are probably many agents and others who can spare such samples," says Mr. Field, "as they will no doubt have no further use for them and would be willing to hand them over in the interests of British trade."

PATENT INFRINGEMENT SUIT

Alleging infringement on their cement-gun patents, the Cement-Gun Company, Inc., of Allentown, Pa., have instructed attorneys to enter suit against the Concrete Construction Co., Ltd., Ottawa, Ont. John A. Pearson, the architect for the Parliament Buildings, specified that the cement-gun be used in fireproofing the structural steel work in the Parliament Buildings, and the contract was awarded to the Concrete Construction Co., who are alleged to have devised a "home-made" apparatus which the Cement-Gum Company says is a direct infringement on their patents for applying sand and cement mortar by means of a compressed air jet.

The largest hydraulic power system in Europe is to be built in Bavaria and operated as a public utility, according to a daily press report. The power is to be obtained from Walchen Lake, in South Bavaria.



Some of the Principal Bridges of the World Compared

(Also see pages 574 and 575 in last week's issue)

From "The Engineer," London, Eng.

PROPORTIONING THE MATERIALS OF MORTARS AND CONCRETES

(Continued from page 8)

Water.—All water used in the tests was taken from the city water mains, the source of supply being Lake Ontario. The chemical analysis is as follows:

	Parts per Million.
Total solids	120.0
Alkalinity (lacmoid) bicarbonates 103; carbonates 2	105.0
Permanent hardness	32.5
Total hardness	137.5
Silicious matter	3.84
Iron oxide, alumina and phosphates	0.17
Lime (CaO)	43.4
Magnesia (MgO)	12.2
Sulfates (SO_4)	18.5
Chlorides	9.0

Composition and Preparation of Test Sands

The grading of the sands used in the tests was not predetermined. In combining the portions retained upon each sieve an effort was made not only to provide a range of gradings comparable with the usual gradings of natural sands, but also to include a few rather irregular or "freak" gradings. Table V. gives the gradings of the test sands. Several of these gradings are shown graphically in Fig. 5.

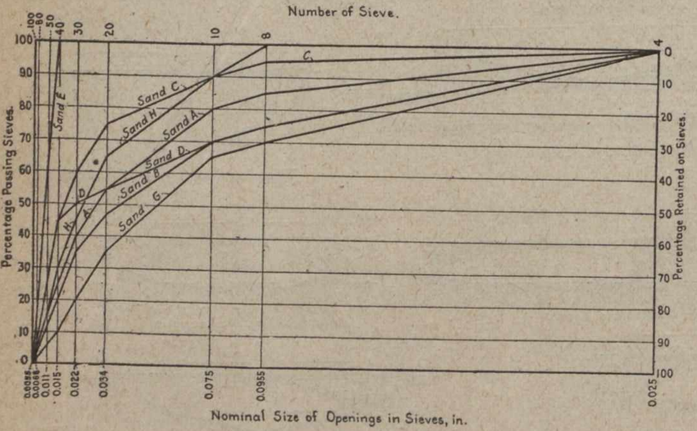


Fig. 5—Grading of Test Sands

Fig. 6, reproduced from photographs of the fractured surfaces of mortar cylinders, gives a general idea of the range of sand gradings used in the tests.

The grading of a test sand having been decided upon and the quantities of the component sizes having been computed and weighed, a uniform composite sand was produced by hand mixing.

As previously mentioned, the sands used in concrete tests were not subdivided into their component sizes.

Test Specimens and Testing

The test specimens made in connection with the various tests were as follows:

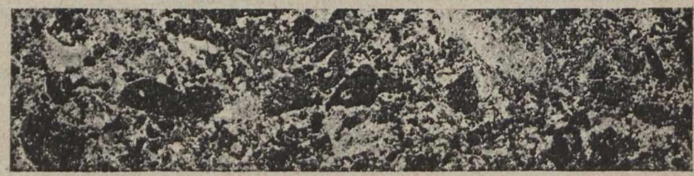
1. For the mortar consistency tests, standard briquettes only.
2. For the mortar strength tests, standard briquettes and cylinders 2 ins. in diameter by 4 ins. long.
3. For the concrete tests, cylinders 6 ins. in diameter by 12 ins. long, and from the mortar content of the excess concrete cylinders 2 ins. in diameter by 4 ins. long.

All test specimens of mortar consistency and mortar strength tests were tested at the municipal laboratory of the city of Toronto. All concrete test specimens and the mortar test specimens made in conjunction with them were tested at the testing laboratory of the Hydro-Electric Power Commission of Ontario, Toronto, Ont. In both laboratories the testing machines used were manufactured by the Tinius Olsen Testing Machine Company, Philadelphia, Pa.

In all compression tests a spherical seated bearing block was used and the ends of the specimens were bedded with sheets of beaver board to secure an even distribution of the load.

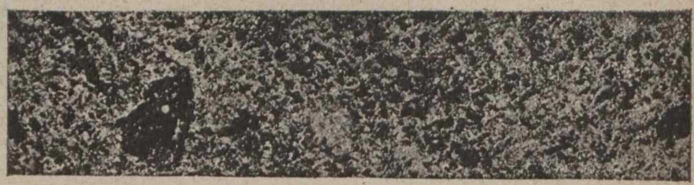
In all mortar tests, except the so-called "consistency tests," four 2-in. cylinders and four "standard" briquettes were tested at each age of 7, 30 and 60 days. Only briquettes were used in the mortar consistency tests. In the concrete tests five specimens were tested at each age.

For all tests the maximum load at ultimate failure was recorded.



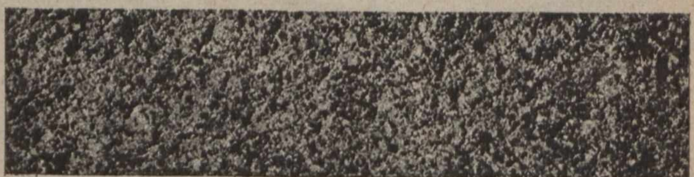
(*a*) Sand A.

(*b*) Sand B.



(*c*) Sand C.

(*d*) Sand D.



(*e*) Sand E.

Fig. 6—Texture of Mortars (× 4)

Preparation of Test Specimens

In the preparation of test specimens a special effort was made to do the work with the greatest dispatch, consistent with the securing of uniform results. The mixing of the materials and the making of test specimens was the work of two operators.

Mortar Proportioning, Mixing and Molding.—Proportioning the cement content of a mortar in relation to the surface area of the sand used is in no way directly comparable with the commonly used method of proportioning by volume the quantities of cement and sand. A "standard" 1 : 3 Ottawa sand mortar, as used in cement testing, was computed to contain 1 g. cement to 13 sq. ins. of surface area. This ratio or proportion was used in many of the tests.

The computed relation of the cement content to the surface areas of several sands assumed to be combined in mortars of 1 : 1½, 1 : 2, 1 : 2½, etc., mix by volume gave a suggestion as to the range of cement content to be used in tests intended to show the relation of strength to the surface area assumed to be covered per unit of weight of cement. This led to the adoption of the proportions, 1 g. cement to 10, 15, 20 and 25 sq. ins. of surface area for such tests.

Table V.—Grading of Test Sands

Sieve Passed and Retained on.	Percentage Retained on Sieve.																
	Sand Letter.																
	A	B	C	D	E	F	G	H	I	J		L	M	N	O	P	Q
P 4 – R 8	15.0	25.0	5.0	25.0	0.0	10.0	30.0	0.0	0.0	8.0	10.0	15.0	8.0	14.0	11.0	11.0	9.5
P 8 – R 10	5.0	5.0	5.0	5.0	0.0	5.0	5.0	10.0	10.0	5.0	6.5	7.0	3.0	8.0	5.0	5.5	5.0
P 10 – R 20	25.0	23.0	15.0	15.0	0.0	18.0	30.0	25.0	25.0	22.0	16.0	22.0	11.5	21.0	16.0	18.0	19.0
P 20 – R 30	15.0	12.0	15.0	5.0	0.0	11.5	15 0	20.0	20.0	10.0	7.0	12.0	7.0	8.0	10.5	13.0	14.5
P 30 – R 40	15.0	15.0	15.0	5.0	0.0	18.0	10.0	15.0	15.0	14.0	16.5	14.0	10.5	12.5	15.5	16.0	20.5
P 40 – R 50	10.0	8.0	20.0	20.0	36.0	21.0	4.0	15.0	15.0	20.5	21.0	15.0	26.0	15.5	18.0	15.0	18.0
P 50 – R 80	10.0	8.0	20.0	20.0	52.0	15.0	4.0	10.0	10.0	20.0	21.5	13.0	32.0	19.0	20.0	15.5	12.0
P 80 – R 100	5.0	4.0	5.0	5.0	8.0	1.5	2.0	5.0	0.0	1.5	2.0	1.0	2.5	2.0	2.0	4.0	1.0
P 100	0.0	0.0	0.0	0.0	4.0	0.0	0.0	0.0	5.0	0.0	0.0	0.0	0.0	0.0	2.0	3.0	0.5
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	101.0	100.5	99.0	100.5	100.0	100.0	101.0	100.0

All mixing of mortar and molding of test specimens was in accord with the Standard Specifications and Tests for Portland Cement and Tentative Specifications and Tests for Compressive Strength of Portland Cement Mortars of the Society for Testing Materials. The consistency of mix used was determined by special investigation. See "Mortar Consistency Tests."

Concrete Proportioning, Mixing and Placing.—The materials for concrete were proportioned as follows:

Cement, 21.12 lbs. per 1,000 sq. ft. of surface area of sand aggregate (1 g. cement to 15 sq. ins. area). For the portion of sand passing a No. 100 sieve the surface area was assumed to be the same as that of an equal portion of material passing a No. 80 and retained upon a No. 100 sieve.

Sand and stone aggregates, 1 part sand to 2 parts stone by volume.

In order to secure greater uniformity these volumes were reduced to a unit weight basis. These unit weights were as follows:

1 cu. ft. sand assumed at 100 lbs.
1 cu. ft. stone assumed at 87 lbs.
1 cu. ft. water assumed at 62.37 lbs.

The water content of the mix was sufficient to produce a saturated, sticky, semi-plastic mortar showing no free water.

It will be noted in the above that in the proportioning of the cement content of the mix no account was taken of the surface area of the stone aggregate. At the time of making the test specimens the approximate areas of broken stone and gravel aggregates had not been determined.

The concrete was mixed by hand. The cement and sand were first incorporated to form a mixture of uniform color. Water was then added to form a mortar, into which the broken stone, after it had been surface moistened, was shoveled and the mixing of the entire mass continued until the particles of stone aggregate were thoroughly coated with mortar.

In the puddling of the concrete in the forms special attention was given to the uniform distribution of the broken stone and mortar. Bars having a diameter of 5/16 to ⅜ in., flattened to form a comparatively thin blade-like end, were used to secure this distribution. Compacting and the exclusion of entrapped air were mainly accomplished by lightly tapping the exterior surface of the forms with a wooden mallet.

From the excess concrete remaining from each batch after the 6-in. cylinder forms were filled, an amount of mortar sufficient to fill twelve 2-in. cylinder molds was secured. The larger stone particles of the concrete were removed by screening upon a ⅜-in. screen.

The forms were removed from the test specimens 48 hours after filling. As soon as removed from the forms the concrete test specimens were marked for identification and placed in storage, where they were moistened three times per day by spraying, until required for testing. The mortar cylinders, upon their removal from the molds, were immersed in water.

Mortar Consistency Tests

The marked influence of the consistency of the mix upon the ultimate strength of mortars renders it especially important that test mortars be made of uniform consistency. The importance of this investigation as a preliminary to the making of tests tending to prove or disprove the validity of the primary theory of the surface-area method of proportioning, is self-evident. Tests were therefore made with the object of developing a means of securing uniformity of consistency in accord with the secondary or consistency theory of this method of proportioning; that is, "The amount of water required to produce a normal, uniform consistency of mortar is a function of the cement and of the surface area of the sand aggregate to be wetted."

It is useless to even outline the preliminary tests made: suffice it to say that only trial or so-called "cut and fit" methods were found to be applicable. As a result of the preliminary tests it was found that "normal," uniform consistency mortars of varying cement content and of varying sand gradings were produced when the quantity of water used in the mix was made equal to (1) that required to reduce the cement to a normal consistency paste, plus (2) an amount equal to the surface area of the sand in square inches divided by 210, that is,

$$\text{Water (cc.)} = \left(\begin{matrix}\text{Weight of cement (g.)} \times \text{Percentage required} \\ \text{to produce "normal" consistency paste}\end{matrix}\right) + \frac{\text{(Total surface area of sand (sq. in.).}}{210}$$

With a water content determined by the above equation each of the five test sands J, K, L, M and N, described under heading "Composition and Preparation of Test Sands" (see Table V.) was used in the preparation of two test mortars having cement contents proportioned 1 g. cement to 10 sq. ins. sand area and 1 g. cement to 15 sq. ins. sand area, respectively. The composition of these mortars is shown in Table VI. Standard briquettes for tension tests were made from each mortar. The strengths of these briquettes are shown in Fig. 7.

Table VI.—Consistency Tests: Composition of Mortars

Sand Letter.	Surface Area per 1000 g., sq. in.	Cement Content 1 g. : 10 sq. in.		Cement Content 1 g. : 15 sq. in.	
		Cement, g.	Water, cc.	Cement, g.	Water, cc.
J.	7151	715.0	193.0	477.0	140.0
K.	7301	730.0	197.5	487.0	143.5
L.	5745	574.5	155.0	383.0	112.5
M.	8811	881.0	238.0	587.5	172.5
N.	6498	650.0	175.5	433.0	127.5

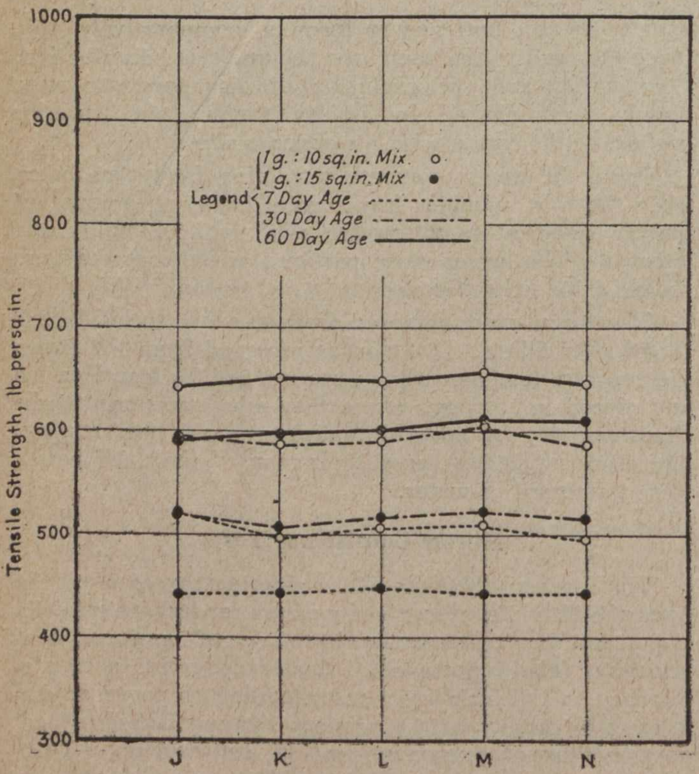


Fig. 7—Consistency Tests—Tensile Strengths of Mortar Briquettes

(Concluded in the next issue.)

Contracts will soon be awarded for the construction of three concrete bridges on the Toronto-Hamilton Highway. The Lewis Construction Co., of Toronto, who were recently awarded the contract for the Bronte bridge by the Highway Commission, have moved the old bridge so that it can be used during the construction of the new one, and have begun work on the new concrete bridge.

SHIPBUILDING PLANT AT HALIFAX

According to public statement made recently by J. W. Norcross, president of the newly incorporated Halifax Shipyards, Limited, within three months the keels of three 10-ton steel freighters will have been laid on their berths. Mr. Norcross says that the expenditure will be between $3,700,000 and $4,300,000. Tenders were called to be in at noon yesterday for the necessary excavating, filling, grading, piling, track-laying, sewers, water mains and concrete wall construction.

The chairman of the board of directors is James Carruthers; president, J. W. Norcross; vice-president and managing director, Roy M. Wolvin; vice-president, M. J. Haney; treasurer, F. S. Isard. S. M. Brookfield, formerly president of the Halifax Graving Dock Company, has been retained by the new company in an advisory capacity. H. M. Brown, of Vancouver, is in Halifax to assist President Norcross with the preliminary details.

"When our plant is completed," says Mr. Norcross, "we will have one of the finest shipyards in America. All of the labor available at Halifax will be used first. Some 3,500 men will be required at the new plant. We will use shipwrights who are now engaged in Halifax and others in Nova Scotia who are proficient in wooden shipbuilding."

WINNIPEG AQUEDUCT BOOKLET

By authority of the administration board of the Greater Winnipeg Water District, a 48-page and cover 6" x 9" booklet has been issued under the title of "Aqueduct Construction Scheme, What It Is and What It Means."

The booklet gives a brief history of Winnipeg's water supply, the reasons leading up to the adoption of the Shoal Lake scheme, the chief features of the design, the progress that has been made year by year in the construction, and a study of the approximate saving to the people of Winnipeg and district by the use of the softer and purer water from Shoal Lake instead of the present supply.

There is also a financial statement showing that the board has issued securities totalling $10,782,112. It is stated that the work will be completed very close to the original estimate of $13,045,000. The booklet contains a number of photographs printed on coated paper and also a supplement showing profile of the aqueduct. The chief engineer of the commission is W. G. Chace, Winnipeg, Man.

CORRECTION

Two minor errors crept into the article on the Chippawa-Queenston power development which appeared in *The Canadian Engineer* for June 20th, 1918. As a matter of record, correction of these errors is desirable. The capacity of the 84-inch Traylor jaw crusher was referred to as being 20,000 cubic yards of crushed stone per day. This was a typographical error, the correct figure being 2,000. On page 546, in the second paragraph of the second column, substitute the word "net" for the word "gross" and substitute "305 ft." for "316 ft."

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Principal Contents of this Issue

PROPORTIONING MORTARS AND CONCRETES

IN proportioning mortars and concretes, does the strength depend only upon the cement-water ratio or upon the relation between the cement content and the surface area of the aggregate? That is the question which is now raised as a result of the experimental work done by Prof. Abrams, of Chicago, and by Capt. Llewellyn N. Edwards, of Toronto. On page 501 of our issue of June 6th, 1918, we reprinted an article by Duff A. Abrams, professor in charge of the Structural Materials Laboratory of the Lewis Institute, Chicago, in which he claimed that the strengths of mortars and concretes for given materials, made up and tested in a similar manner, depend upon nothing but the relation between the amount of water and the amount of cement in the mix. In his article Prof. Abrams says:—

"It is seen that for given concrete materials the strength depends upon one factor only—the ratio of water to cement. Equations which have been proposed for this purpose contain terms which take into account such factors as quantity of cement, proportions of fine and coarse aggregate, voids in an aggregate; but they have carefully omitted the only item which is of any importance, the water."

Prof. Abrams plots a curve showing compressive strengths in pounds per square inch in direct relation to the cement-water ratios. The equation of this curve he gives as $S = A/B^x$, where S is the compressive strength of the concrete and $x = W/C$ (W being the volume of water and C the volume of cement), A and B being constants whose values depend upon the quality of the cement, the age of the concrete and the curing conditions. For the conditions of the tests conducted by Prof. Abrams, he states that $A = 14{,}000$ and $B = 7$.

"Other tests made in this laboratory," says Prof. Abrams, "have shown that the character of the aggregate makes little difference if it is clean and not structurally deficient. . . . In certain instances a 1 : 9 mix is as strong as a 1 : 2 mix, depending only on water content. The strength of the concrete responds to changes in water, regardless of the reason for these changes."

About the same week that Prof. Abrams announced the results of his tests, Capt. Edwards, supervising engineer of bridges of the Works Department of the City of Toronto, forwarded to the American Society for Testing Materials a paper describing a series of tests which he had made since November, 1917. The results of those tests do not altogether agree with the laws promulgated by Prof. Abrams, and the difference is of decided interest to users of mortars and concretes and will no doubt lead to much discussion, and probably to further investigation by Prof. Abrams, who is an earnest and indefatigable worker; he has accomplished research of unquestionable value and will be the first to desire to clear up any existing doubts as to the best method of proportioning.

Capt. Edwards agrees with Prof. Abrams that a plastic mix is essential and that excess water in an oversaturated mortar or concrete mix exerts a decidedly weakening effect upon the cement matrix by producing a change in the physical structure of the matrix which tends to destroy the cohesion existing between particles of cement and the adhesion existing between the cement and the aggregate; but as a result of his extensive tests, Capt. Edwards claims that the strength of mortar containing a given cement and sand but varying for the different mixes in the proportions of these ingredients, is directly proportional to the relation existing between the cement content and the surface area of the aggregate.

He assumes the use of enough water for a "normal," uniform consistency, and that there will be no excess of water, but he does not recognize thaa the compressive strength depends only upon the cement-water ratio. In Prof. Abrams' article there is no evidence that he takes into account the volumetric composition of the sand, to which Capt. Edwards attaches the maximum importance on account of the variable sand areas resulting from differences in the sizes of particles.

Capt. Edwards makes it clear in his paper that he fully realizes the importance of obtaining correct cement-water ratios, although not attaching the sole and exclusive importance to this item that does Prof. Abrams. In fact, Capt. Edwards has gone a step further than other investigators in showing the actual cause of weakening by the use of too much water. Prof. Abrams, Capt. Edwards and others have previously shown by many tests that an excess of water has a marked effect upon cement products, but nobody previously made it so clear as to the cause of this effect. Capt. Edwards' experiments show that the cause is a structural change in the cement matrix, which is the only material of the mix that has any cementing value.

Capt. Edwards claims that his method provides a means for a more thorough investigation of the functions of sand and stone aggregates and for more complete development of the true values of all component materials of mortars and concretes. The relations between the materials as established by Capt. Edwards will appeal more readily to the average engineer than will Prof.

Abrams' formula. The latter is so very revolutionary; while the former is along the lines of present practice, only establishing a more scientific basis for the proportioning of the materials.

The results of Capt. Edwards' tests confirm the popular idea that, in both mortars and concretes, for a condition of maximum strength, there should be sufficient neat cement to coat entirely every particle of aggregate; and in concretes, sufficient mortar to coat every stone. Capt. Edwards has established his claims upon a very scientific basis. A great amount of tedious work was involved in his tests. For instance, he had to count the number of particles in a given quantity of various sands, and in more than one direction his work lay along lines which greatly taxed the patience, the nerves and the eyes. He is to be congratulated upon the results, which appear to be of much value. His paper is not solely confined to theoretical discussion, nor yet to practical tests. It embraces relations between design and construction which are frequently overlooked, and shows clearly how engineers and contractors can apply his method in actual practice. Unless Prof. Abrams' theory supplants that of Capt. Edwards, it is very likely that the materials of nearly all mortars and concretes will be proportioned along the lines suggested by Capt. Edwards rather than by volume, as has been done at least since 1827 and possibly since the "constructive" Roman period. As Prof. A. H. Heath is given credit by Capt. Edwards as being the pioneer or perhaps the originator of his method of proportioning, we would suggest that his method be known as the Heath-Edwards method of proportioning.

"The volume method, as commonly used," says Capt. Edwards, "is not only illogical and unscientific, but also unfair to the development of the true value of the materials entering into the composition of both mortar and concrete. The ideal mortar contains a proportion of cement sufficient to develop the full strength of the particles of sand aggregate; while the ideal concrete contains a mortar component, in itself ideal, which will develop the full strength of the particles of stone aggregate. There should be no place in engineering for guesswork and empiricism whenever scientific determination is possible."

PERSONALS

Sidney M. Johnston, has been appointed temporary city engineer of Stratford, Ont.

F. M. Mahard, superintendent of construction of the American Can Co., Montreal, has resigned to become designing engineer for Monks & Johnson, Boston.

Lieut.-Col. Ibbetson Leonard, of London, Ont., has been awarded the D.S.O. He is a member of the firm of E. Leonard & Sons, boiler and engine makers.

J. A. Burnett, electrical engineer, Grand Trunk Railway System, has received an appointment as technical assistant to the British War Mission, Washington, D.C.

James C. Wardrop, engineer on the staff of the Department of Works, Hamilton, Ont., has resigned, and will in future be associated with F. R. Warren, architect.

Major Graham Bell, financial comptroller of the Department of Railways and Canals, Ottawa, is to supersede A. W. Campbell as Deputy Minister of Railways and Canals.

Hon. C. C. Ballantyne, Minister of Marine and Fisheries, will shortly go to Great Britain to take up naval and other maritime problems with Admiralty and shipping authorities in that country.

L. H. Harza, formerly chief engineer in charge of design and construction of the 20,000-h.p. Canadian hydro-electric development at Sault Ste. Marie, Ont., is now in Jacksonville, Fla., taking charge of the proposed concrete shipyard.

Lieut. A. Rupert Neelands, M.C., who graduated from the University of Toronto in 1906, has been awarded the Military Cross. Previous to enlisting with the Canadian Engineers and serving in France, Lieut. Neelands followed his profession at Port Hammond, B.C.

James, Loudon & Hertzberg, consulting engineers, Toronto, have been appointed on behalf of the Public Works Department of Canada to valuate the equipment formerly used by the Confederation Construction Co. on Contract 3 of the Welland Canal, preparatory to purchase of the equipment by the government.

Lieut. Harold J. Mackenzie, an honor graduate of the University of Toronto, in Applied Science, has been awarded the Military Cross. He enlisted with the Canadian Engineers and after being attached for some time to the training depot at Ottawa, went to France with the 1st Tunnelling Co., Canadian Engineers in July, 1916.

Lieut.-Col. Charles H. Mitchell, of Toronto, was included in the King's birthday honors, being made a Companion of the Bath for services in Italy. Prior to enlistment, Col. Mitchell was senior member of the consulting engineering firm of C. H. & P. H. Mitchell, Toronto. He has held various high positions in the Intelligence Service of the British Army, practically since the beginning of the war, and has been decorated a number of times and frequently mentioned in despatches.

J. R. M. Fairbairn has been appointed chief engineer of the C.P.R'y Co. replacing John G. Sullivan, chief engineer, who is retiring to enter private practice. Mr. Fairbairn was born in Peterborough 45 years ago. He entered the University of Toronto, and graduated in 1893. Following a short private practice in British Columbia, he joined the C.P.R. as a draughtsman at Winnipeg, which position he held for two years. He was then made resident engineer at Place Viger, Montreal; assistant engineer, Toronto; assistant engineer of maintenance of way, Montreal; division engineer, Toronto, and engineer of maintenance of way, Montreal. In February, 1911, he was made assistant chief engineer.

OBITUARIES

Dr. Jasper Douglas, a Canadian, who was for many years president of the board of directors of Phelps, Dodge & Co., copper mine owners, died recently at New York at the age of eighty-one. Dr. Douglas was a graduate in mining at Queen's University and early in his career practiced at Quebec. Since 1875 he had been residing in New York, where he had charge of extensive copper interests. He was a member of the American Institute of Mining Engineers and also of the North of England Mining Engineers' Society.

John H. Kerr, secretary of the Canadian Westinghouse Company, of Hamilton, died on Monday, the 24th ult. Mr. Kerr was a native of Pittsburg, Pa., and had been engaged with the Westinghouse companies for many years. He began with the parent company and also assisted in the organization of the French Westinghouse Company. In 1901, when the British Westinghouse was founded, he went to Manchester to organize the factory system. In 1903, on the formation of the Canadian company, he was transferred to Hamilton.