PAGES

MISSING

The Canadian Engineer

A weekly paper for Canadian civil engineers and contractors

WATER POWERS OF NEW BRUNSWICK AND PRINCE EDWARD ISLAND

AN INTERESTING ACCOUNT OF THE WATER POWER POSSIBILITIES OF THE TWO PROVINCES, TOGETHER WITH FACTS CONCERNING ACTUAL DEVELOPMENT WORK DONE.

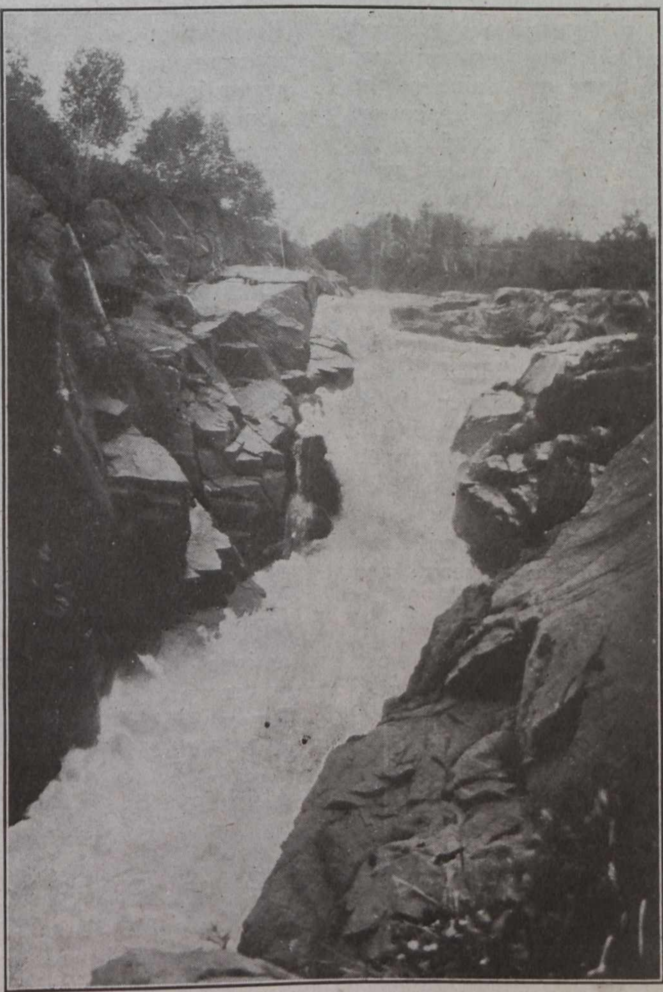
By K. H. SMITH, A.M.Can.Soc.C.E.
Resident Engineer, Dominion Water Power Branch.

THE province of New Brunswick is roughly rectangular in outline, with its longest dimension lying north and south. Its south and east sides are bounded by the Bay of Fundy and Northumberland Strait, with the exception of a neck of land about 15 miles wide joining it on its southeast corner to the province of Nova Scotia. It adjoins the State of Maine on its west side, and the province of Quebec on the north.

It is, roughly, 210 miles long by 140 miles wide and lies within north latitude 45 degrees and 48 degrees and longitudes 64 degrees and 68 degrees west from Greenwich. Its total area is 27,985 square miles, of which 181 square miles is water surface. The total population is about 360,000.

Topography.—The country is generally undulating, with the exception of the east coast, which for some distance inland is relatively flat. The Bay of Fundy shore is bold and rocky, and not far inland a prominent ridge, generally from 500 to 1,000 feet high, parallels this shore. With the exception of this ridge, the southeast half of the province lies at an elevation from 100 to 500 feet above sea level, with a number of broad, flat valleys. The northwest part of the province is generally from 500 to 1,000 feet above sea level, with some areas from 1,000 to 2,000 feet. The highest elevation in the province, about 2,600 feet, is in the extreme northwest corner.

On the whole, the topography of the province may be considered as quite mature, giving evidence in the undulating surface and broad flat valleys of long subjection to the various eroding elements. In those areas underlain by rocks of the harder varieties as noted below, the topography is naturally more rugged.



Grand Falls, N.B.

Geology.—The greater part of the geological structure of New Brunswick consists of sedimentary rocks of the Palaeozoic age, though considerable areas of granite are also in evidence.

The geological formation underlying the plateau extending along the southern part of New Brunswick and parallel to the Bay of Fundy is composed of a variety of rocks, chiefly granites and granite-gneisses. There are also three relatively large areas of granite in the central part of the province, lying in a general southwest and northeast direction. Between this granite area and the igneous plateau extending along the Bay of Fundy is a large triangular area composed mostly of the Pennsylvania series of the carboniferous or upper Palaeozoic period, with some areas of the Mississippian series immediately adjoining the igneous plateau. By far the greater part of the northwest half of the province consists of unclassified rocks of the Devonian, Silurian and Ordovician periods of the Palaeozoic age.

The mineral resources of the province are probably not large. Gypsum, petroleum, natural gas, coal and antimony are being produced in commercial quantities, while iron, copper, manganese and graphite, have been mined at various times.

Lumbering and Fisheries.—The province of New Brunswick is well timbered and lumbering in all its branches is an outstanding industry. Practically all the logs are driven to their destination by way of the existing rivers, and this fact, as well as consideration for the vested interests, must be given due weight in all power investigations and estimates, and particularly in the design of all structures for hydro-electric purposes.

The fisheries are also important, both from the commercial and sporting aspect, and it is absolutely essential that adequate fishways should be provided at all structures placed in the rivers.

Climate and Precipitation.—As a rule, severe winter conditions exist for several months in the year, particularly in the interior of the province, with an abundance of snow. Ice conditions of all kinds are, therefore, quite severe and excessive floods in the spring months are to be expected.

Precipitation is not excessive, and probably for the greater part of the interior of the province does not exceed 35 inches per year as an average.

River Systems.—The outstanding features of the province from a water-power standpoint can best be understood from a discussion of a few of the more important rivers.

The St. John River.—This river and the basin it drains is one of the outstanding features of the Maritime Provinces. By far the largest river in this part of the Dominion or the United States, it has a total drainage area of 26,000 square miles and a total length of about 450 miles. It is navigable for craft of commercial size from St. John to Grand Falls, a distance of over 200 miles; for 70 miles it forms the boundary between New Brunswick in Canada and the State of Maine in the United States, and for the first 90 miles of its course lies wholly within the State of Maine. Its broad valley, extending through the province for some 280 miles, with an average width of 100 miles, contains about two-thirds of the total population of the province, and offers excellent opportunities for agriculture. The river itself throughout this distance is a main thoroughfare for floating timber.

From the Reversible Falls at St. John, an unique phenomenon caused by a variation in the tide of about 30 feet at this point, to Fredericton, a distance of 50 miles, the valley is well wooded and sparsely settled. The lower part is quite rugged, while above that much flat intervale land occurs. The more important part of the valley lies above Fredericton, where rolling uplands and wide, cultivated valleys occur. Above Grand Falls and around the headwaters of all tributary streams the country is thickly wooded.

The only natural water-power site on this river lying wholly within New Brunswick, and, indeed, the most outstanding water-power in all the Maritime Provinces, is at Grand Falls. This site is situated in the north-west corner of the province, not far from the Maine border, and about 200 miles from St. John, the main centre of population and industry. A direct drop of 74 feet occurs at this point, while it is possible to obtain a working head of 130 to 140 feet. The total tributary drainage area is 5,280 square miles, with considerable storage available.

It is evident that from the standpoint of magnitude and natural advantages this site compares very favorably with many other sites in Canada, either developed or proposed.

A number of other sites on the river, notably the Pokiok site and Meductic Falls, have been considered from time to time, but they offer few natural advantages. Several more promising sites exist on the upper waters of the river in the State of Maine, as well as one or two international sites.

The Tobique, one of the larger tributaries of the St. John, with a drainage area of 1,728 square miles, flows near its mouth through a restricted, gorge-like channel, which offers opportunities for creating an artificial head.

The Aroostook, another, and the largest tributary of the St. John, lies almost wholly in the State of Maine, though one of the larger power sites of the Province of New Brunswick exists on this river just over the Canadian boundary. Practically the whole of the drainage basin of the river (2,280 square miles) is tributary to this site, where a head of 77 feet is available, and which is partially developed at the present time by the Maine and New Brunswick Electric Power Company. Practically no storage is available.

A considerable amount of valuable run-off data is available for this river, as well as some of its tributaries, and all such data to the end of 1911 may be found in the 1911 report of the Commission of Conservation, Canada, on the "Water Powers of Canada." Data since that time are published in the Water Resources papers of the United States Geological Survey, part of which information is obtained from private parties interested at Grand Falls.

The St. Croix River.—This river system forms about one-half of the boundary between the Province of New Brunswick and the State of Maine, and is entirely an international stream. Its total drainage area above tidewater is 1,470 square miles, and it has a lake area of about 160 square miles.

From the standpoint of size, natural regularity of flow, with possibilities of further artificial regulation on its extensive lake system and facilities for development, it is one of the most important power rivers in this district. Considerable development on it has already taken place, most of which, however, is on the American side. The St. Croix Paper Company, a United States corporation, has a plant at Woodland with about 13,500 horse-power installed, and another at Grand Falls, where 8,000 horse-power is installed, with provision for the installation of another 4,000 horse-power. Existing or proposed developments are generally of a low-head type, taking advantage of low falls or rapids. At Woodland and Grand Falls, for example, the operating heads are 47 feet and 49 feet, respectively, while a number of other sites exist with possible heads from 8 to 40 feet.

Considerable information is available in connection with this river, due to the investigations of the International Joint Commission dealing with diversions for power purposes and a complete power report by F. A. Roper, Major Corps of Engineers, United States Army, and William J. Stewart, Department Naval Service, Canada, is available in the 1915 Report of the Maine Public Utilities Commission, Volume 2, recently issued. The United States Geological Survey publish run-off data for the river from year to year, as also the State of Maine Public Utilities Commission.

Nepisguit River.—This river offers the only favorable opportunities for power development of commercial magnitude in the eastern part of New Brunswick. At Grand Falls, on this river, there is a tributary drainage area of 644 square miles, with an available head of about 125 feet, most of which is concentrated in the falls itself.

A number of rapids with high, rocky banks exist in the lower reaches of the river, where heads from 25 to 60 feet might be concentrated by suitable dams.

It is worthy of note that a large deposit of iron ore exists in the immediate vicinity of Grand Falls. An excellent railroad, about 20 miles long, connects this iron mine with the main line of the I.C.R., but at present the mine is not operating.

The South-West District.—This part of the province is considered separately, because it is the only part of the province which has those physical characteristics which would seem to indicate promising water-power possibilities, such as lakes for storage purposes and rocky river basins. It includes the St. Croix River, already discussed, the Magaguadavic, the Lepreau, the Musquash Rivers and a number of smaller streams.

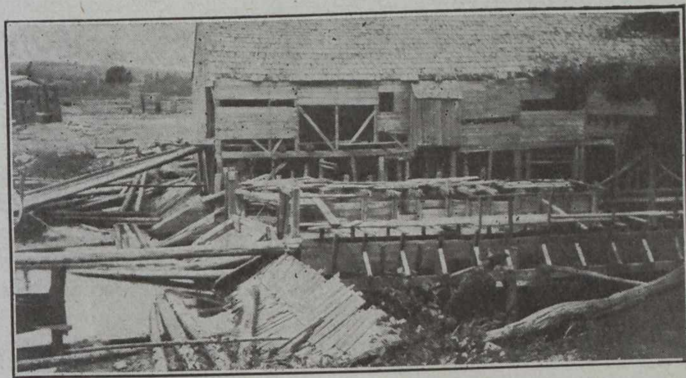
Practically all the rivers in this district admit of developments of one kind or another. The available heads are low, and in general the drainage areas are small, though some opportunities exist for artificial regulation of flow. Apart from the St. Croix River, which has already been discussed, and is shown to be of considerable importance, the only other outstanding river in this district is the Magaguadavic. At St. George Fall, which is partly developed by a pulp company, with 3,000 horse-power installed, there is a tributary drainage area of 688 square miles with a head of 45 feet. Several other sites exist on this river with slightly smaller drainage areas and heads from 16 to 20 feet. Storage can be provided on a number of lakes in the basin.

Other Rivers.—The Miramichi, with its various branches, the Restigouche, Upsalquitch, Canaan and Salmon Rivers are all of large size, but in general have no pronounced falls or rapids of appreciable magnitude, and, furthermore, are absolutely devoid of lakes which might be used for artificial regulation. The Restigouche and Miramichi are tidal for considerable distance from their mouths. In a few cases, notably on the Miramichi and Upsalquitch, and generally near the upper waters of these streams, low falls and flat rapids occur, with high, rocky banks, where heads of some magnitude might be created by high dams. Such developments are, of course, expensive.

As a general thing, even on the smaller streams, few falls or rapids occur, and head for small developments, a number of which are in operation for local purposes, must be obtained almost entirely by dams. Furthermore, as in the case of the larger rivers, practically no lakes or storage basins exist on these small streams.

Sackville River—A Typical Log-driving Dam.

General.—The conspicuous absence of lakes for natural or artificial regulation, combined with only a moderate rainfall, and that not evenly distributed, is a serious handicap from a water-power standpoint. Furthermore, no opportunities for creating high head developments to thereby compensate somewhat for small flows at certain times of the year are available, and even moderate or low-head sites are not plentiful or well distributed. And yet at Grand Falls on the St. John, Aroostook Falls and Grand Falls on the Nepisguit, the outstanding water-powers entirely within the province, New Brunswick possesses water-power assets of no small magnitude which will undoubtedly become of more and more importance. For the present, the size of these powers is somewhat of a disadvantage. With the possible exception of Grand Falls, they are too small to attract large users of power, such as chemical industries, to their immediate vicinity, and, on the other hand, are too large and too far removed from present industrial centres to permit of immediate development.



Sackville River, N.B., Typical of a Great Number of Small Mills Found in Any Part of the Country.

Undoubtedly the time will come when the increase in existing industrial and railway centres and the creation of new centres, aided by the knowledge of water-powers ultimately available, will warrant long distance transmission lines from these sites.

It is certainly most urgent that adequate investigations should be undertaken leading to comprehensive administration, and it is just possible that it will be shown that, in relation to its present and prospective needs, New Brunswick is better off for water-powers than has generally been supposed.

Prince Edward Island.—The Province of Prince Edward Island certainly has no water-powers of commercial magnitude except in a very small way. It is solely an agricultural community, and has many small streams which doubtless could be used to advantage by individual farmers through the medium of small hydro-electric developments installed by local labor. In these days, when every effort is being made to stimulate agricultural production, apparently a good purpose could be served by indicating to the farmers just what might be done with the small brook which is to be found on most farms.

There are quite a number of small water-driven mills in operation, and in the early days, before the land was entirely cleared of timber, many sawmills were driven by water-power.

The area of the Island is 2,184 square miles, with a population of about 94,000, which shows a larger population per unit of area than any other province in the Dominion. The length from end to end is 130 miles and the width varies from 2 to 30 miles. Practically all the land is cultivated, and the highest elevation is only about 300 feet above sea-level. The average annual precipitation is from 36 to 40 inches.

Tidal Power.—The enormous energy exhibited by the great sweep of the Bay of Fundy tides, with an ordinary range in height of 30 to 40 feet, has always aroused the interest of practical men. A number of small mills throughout the Maritime Provinces are run intermittently at periods of low tide with water stored in small reservoirs at high tide by means of flap gates inserted in the impounding dams. So far, this is the only practical use made of the energy in the tidal flow.

Various schemes have been proposed to use this tidal energy in a commercial way, and some years ago a somewhat ingenious scheme was patented by a local surveyor. Recently, new interest has been given to the whole problem, due to investigations carried on by officials of Acadia University and a proposed new type of current motor. These investigations are still under way, but so far as is known neither in these investigations as carried out to date or in any previous investigations has any scheme been evolved which can successfully compete in a large way with other and immediate sources of power. Tidal energy, however, offers a rich field for investigation, and the time may come when decreasing resources of coal, wood, gas or oil will warrant the necessary expense to derive power from these sources.

NEW STANDARD SPECIFICATIONS ISSUED BY ONT. R'Y AND MUNICIPAL BOARD.

By David A. Molitor, M.Am.Soc.C.E.,
Formerly Designing Engineer, Toronto Harbor Commission.

NEW standard specifications for bridges, viaducts, trestles and other structures have just been issued by the Ontario Railway and Municipal Board. The 1910 standard specifications are superseded by the 1916 specifications which have been prepared to meet the requirements of the latest practices in bridge engineering. For instance, the growing demand for modern type movable bridges was given careful consideration.

To facilitate the work of designing engineers, all matters pertaining to the design of steel bridges, of both fixed and movable types, are treated in Part 1, and amplified by thirteen data sheets in Part 8.

Part 2 deals with the "Quality of Materials," covering all material requirements and tests, while "Manufacture and Workmanship" is treated in Part 3. The clauses pertaining to "Field Erection and Painting" are given in Part 4.

Thus, all the requirements governing the construction of a fixed or movable steel bridge are conveniently grouped under Parts 2, 3 and 4, comprising a complete set of construction specifications to accompany the plans whereon all matters of design have been previously determined in accordance with the requirements of Part 1.

A similar arrangement was adopted in Part 5, dealing with concrete bridges, where the clauses on design are again separated from those governing the construction.

Part 6, relating to stone masonry, will not be used frequently owing to the general adoption of concrete and reinforced concrete in modern bridges. However, the use of stone masonry should not be discouraged, and will continue to occupy a limited field, chiefly for aesthetic reasons.

Timber trestles are treated in Part 7. They still find frequent use as temporary and semi-permanent structures, especially on new railway lines.

It is desirable to discuss a few of the new designing requirements chiefly for the purpose of illustrating their justification and application. It will be found upon reading sections B, C and D of Part 1, that the new specifications include some new departures on the subjects of impact, combination of stresses, and allowable unit working stresses.

The method of treating impact stress employed in the 1910 specifications was cumbersome and illogical. According to that method the impact stress was expressed as a function of both dead and live load stresses in each member, and this was further complicated by a factor for increasing the live load for all spans not exceeding 80 ft. While the live load stresses can be definitely found for any given case of truss and loading, the dead load stresses are never definitely known until the structure is completely designed in all details and its weight finally computed. Hence the impact stresses also remain in doubt until the dead load stresses are fixed. This involves the extra labor of adjusting the sections of members to satisfy two variables. Furthermore, the method cannot be harmonized with the results of actual experiments. The article by E. H. Darling, in *The Canadian Engineer* of April 13th, 1916, devotes considerable space to a discussion of this formula, so that no further comment seems necessary.

The impact formulae given in the new specifications depend only on the loaded length producing the stress, so that the impact stress becomes a fixed function of the live load or live load stress, independent of the dead load. This makes it possible to compute the live load and impact stresses for each member, without first knowing the dead load stresses, and subsequent modifications or adjustments in the dead load will not alter the impact stresses. Especially for combination loadings does this method offer many advantages in practical designing.

As to the accuracy of these formulae, it may be said that the first gives safe values in all cases of steam railway bridges and is based on the elaborate experiments made in recent years by the American Railway Engineering Association. The formulae for electric railway and highway bridges undoubtedly err on the side of safety and, in the absence of experimental data, are based entirely on judgment, using the formula for steam railways as a guide.

The formulae for steam and electric railways were evolved by the writer in August, 1911, and the one for highway bridges was proposed by Dr. Waddell in his "De Pontibus," page 369. Dr. Waddell strongly advocates the general adoption of the writer's formula for steam railways in his recent book on Bridge Engineering, 1916.

It should be stated that the curve represented by an impact formula is an enveloping curve of all experimental data on bridges for one specific class of loading, as for example, steam railway trains. This is because impacts, even for the same structure, vary from zero to a maximum, and only the maximum value for each span will govern the safe design.

The subjects of stress reversals and combination of stresses have received widely different treatment in the past, and the 1910 specifications were rather misty on these points.

The clauses under section C, Part 1, of the new specifications cover all cases of combination of stresses of the same or opposite character for all conditions of loading. These should be self-explanatory, but the following is given to show the exact purport of the two last clauses of this section:—

A structure designed for unit stresses approaching the elastic limit simultaneously in all its members, for double the live load plus impact, would obviously have a factor of safety greater than two, throughout; would be uniformly safe; and would be considered a prudent design. The two clauses in question are intended to accomplish this end in every bridge design.

Each bridge possesses some members in which the unit stress resulting from overload, increases at a greater rate than the rate of increase in the load applied. All members, in which the dead load stress is of opposite sign to that of the maximum live load stress, belong to this class; likewise all members, the maximum live load stresses of which are produced by a partial loading of the span.

The total stresses in the chord members and end posts are directly proportional to the total loads covering the entire span length, hence to double the live load will increase the unit stresses in these members by a constant percentage, always less than 100 per cent., so long as the dead load stress is not zero. This same overload will generally increase the unit stresses in the web members in a much greater ratio than for the chords, hence the following two clauses in the specifications:—

"Members subjected to dead and live load stresses of the same character and in which the maximum live load stress is produced by the maximum live load covering the whole span, shall be proportioned for the sum of the dead load, live load and impact stresses on the basis of the allowable unit stresses f given for tension or compression, as the case requires. This applies generally to chords and end posts. Then, with the section so obtained for the middle chord member, and a total stress $S' = D.L. + 2 \times L.L. (1+I)$, find the unit stress f' resulting from 100 per cent. overload in the live load; and also, find the factor $k = f'/f$ (always less than 2) which represents the ratio of increase in the unit stress f due to 100 per cent. overload. This factor k will necessarily be a constant for all members of this class, and is to be used as directed in the following paragraph.

"Members subjected to dead and live load stresses of the same or opposite character and in which the live load stress is produced by a partial loading of the span, as for web members generally, shall be proportioned for the algebraic sum of the dead load stress and twice the live load stress plus impact, using a unit stress kf, where f is the allowable unit stress in tension or compression, as the case requires. Where a reversal in live load stress is possible, the sectional area should be computed separately for each combination with the dead load stress and the larger area will govern the design."

A structure so designed will have a uniform factor of safety well within the elastic limit for an overload of 100 per cent. in the live load; and will have a minimum factor of safety of two on the elastic limit, for the assumed live load. The unit stresses for the overloaded condition will thus approach the elastic limit for short spans wherein the dead load stresses are small compared with the live load stresses, while for long spans with excessive dead loads the effect of the overload is less severe.

The following example will illustrate the method of finding the required sectional areas of members after the dead and live load stresses are known:—

Example.—Given the *d.l.* and *l.l.* stresses in a single-track steam railway bridge of 452-ft. span, with 9 panels (intermediate span of Memphis Bridge). The negative sign indicates compression. Required to section the members. The impact formula is $I = \dfrac{165}{L+150}$, where L is the loaded length producing the *l.l.* stress.

Top Chord, central section U_6U_8 (by clause 1).

Dead load stress	= — 1,200,800 lbs.
Live load stress	= — 889,500 lbs.
Impact stress for I = 0.275	= — 244,500 lbs.
Total stress S	= — 2,334,800 lbs.

Allowable unit stress $f = 17{,}000 - 60\dfrac{l}{r} = 15{,}650$ lbs.

Hence, area required $= \dfrac{2{,}334{,}800}{15{,}650} = 149.3$ sq. ins. gross.

This is as per clause 1, for a top chord member.

Now find $S' = D.L. + 2 \times L.L. (1 + I) = 1{,}200{,}800 + 2 \times 889{,}500 \times 1.275 = 3{,}468{,}800$ lbs.; which represents 100 per cent. overload, and $f' = -\dfrac{-3{,}468{,}800}{149.3} = -23{,}260$ lbs., whence $k = f'/f = \dfrac{23{,}260}{15{,}650} = 1.485$.

Centre diagonal U_8M_7 (by clause 2).

Dead load stress	= — 20,100 lbs.
Live load stress	= — 111,900 lbs.
Impact stress for I = 0.408	= — 45,700 lbs.
Total stress S	= — 177,700 lbs.

Allowable unit stress $f = 16{,}000 - 80\dfrac{l}{r} = 10{,}675$ lbs.

$kf = 1.485 \times 10{,}675 = 15{,}850$ lbs.

Total stress S' with 100 per cent. overload = — 335,300 lbs.

Hence, area required $= \dfrac{335{,}300}{15{,}850} = 21.15$ sq. ins. gross.

The area required without the consideration of overload would have been $\dfrac{177{,}700}{10{,}675} = 16.64$ sq. ins. gross, for which the 100 per cent. overload would have produced a unit stress of $\dfrac{335{,}300}{16.64} = 20{,}150$ lbs. per sq. in., whence $f'/f = \dfrac{20{,}150}{10{,}675} = 1.89$.

This shows that, for double the live load stress, the unit stress in the top chord was increased 1.485 times, and for the post U_8M_7, the same overload would have increased the unit stress 1.89 times on the basis of 16.64 square inches of area, while the actual area required for this post by clause 2 is 21.15 square inches. This same post also receives a counter stress in tension equal to the compression, but, as the dead load stress is compressive, the counter stress did not give the maximum gross area required.

This illustrates how the above specification clauses operate to produce a structure of uniform strength throughout up to the practical limit of safety for the structure as a whole, which is at the elastic limit of the material. This method, therefore, accomplishes in a systematic manner everything necessary to safeguard the design of any member without resorting to crude and illogical devices which are usually wasteful and do not always accomplish the desired result.

The actual labor involved by this method is not excessive as might seem from the above figures, all of which are not required, but are given to show the nature of the results.

As to the allowable unit working stresses given in section D, Part 1, these are believed to represent the best modern conservative practice, and are all based on static load equivalents, assuming that all dynamic effects have been commuted into static effects by impact additions. This applies also to the working stresses given for bridge timber, concrete, masonry and foundations.

The straight line column formula was adopted, as representative of the latest conclusions on column tests, to supersede the Gordon-Rankine formula which is more complicated and is in no sense preferable within the allowable limits of *l/r*.

As to the general scope and completeness of the new specifications, the reader can best judge that for himself.

[NOTE—These specifications were prepared for the Ontario Railway and Municipal Board by Prof. Molitor and H. W. Middlemist, consulting engineer for the board. Copies of the complete specifications can be obtained upon application to H. C. Small, secretary, Ontario Railway and Municipal Board, Parliament Buildings, Toronto.—EDITOR.]

SPECIFIC GRAVITY OF NON-HOMOGENEOUS AGGREGATES.*

By Prevost Hubbard and F. H. Jackson, Jr.

THE object of this investigation was to study methods in common use, or those which gave promise of being most satisfactory, for the determination of apparent and true specific gravity of mineral aggregates, with a view to ascertaining if possible what method is most generally applicable to all classes of materials, and also the most accurate. The methods studied were as follows:

1. The ordinary displacement method as conducted by the U.S. Office of Public Roads and Rural Engineering;
2. The Chapman method for single specimens;
3. The Goldbeck method;
4. The Hubbard-Jackson method;
5. The Chapman wire-basket method;
6. The Le Chatelier method for fine aggregates; and
7. The Jackson method for fine aggregates.

The determination of the specific gravity of material composed of mineral particles is not ordinarily considered a difficult matter, and until recently but little attention has been paid to the subject in so far as non-bituminous road and paving materials are concerned. Certain discrepancies in results obtained in the laboratories of the U.S. Office of Public Roads and Rural Engineering have, however, indicated to the authors that the entire subject of the specific gravity of these products should be investigated with a view to ascertaining, if possible, what method is most generally applicable to all classes of materials, and also the most accurate.

The method most commonly used in determining the specific gravity of such substances as rock is to first weigh a specimen of rock in air and then in water, and from these results to calculate the specific gravity by the usual formula applied to what has been termed the displacement method. It is, of course, evident that unless the average of a large number of determinations upon different specimens is taken as representing the specific gravity of the material, any method employing a single specimen must, of necessity, be accurate only for material which is of a homogeneous nature. Certain rocks, as well as many slags, gravels, etc., are composed of non-homogeneous material, so that a single small specimen can only represent by accident the average of the material as a whole.

In addition to lack of homogeneity in the material itself, there must also be considered lack of homogeneity in the size of particles present in a given lot of material, such as crushed rock, crushed slag and gravel. Both kinds of non-homogeneity may exist in a sample of material whose specific gravity is desired. In such case it may be found that in general all particles above a certain size are more or less alike in character, and that particles below this size vary greatly from these larger particles, as well as among themselves.

Another factor which must be considered is the difference which often exists between the specific gravity of a mass of material and the specific gravity of the substance of which the mass is composed. For instance, it is evident that a rock which is sufficiently porous to absorb water, as almost all rocks do to a greater or less extent, must of necessity when dry contain spaces filled with air. The specific gravity of the mass, then, including the air spaces, is less than that of the substance of which the mass as a whole is composed. The term "apparent specific gravity" has been applied in the first instance and "true specific gravity" in the second. It is evident that the apparent specific gravity may be less than or equal to the true specific gravity, but can never exceed it.

It is often desirable to ascertain the apparent specific gravity of a material. When this is so, it would then seem reasonable to suppose that the most accurate determination of apparent specific gravity could be obtained from determinations made upon particles of the material as it is purchased and used. The reason for this is, that as the size of particles is reduced the proportion of voids or air spaces in the structure of the material becomes reduced, for if reduction were carried to the point of destroying the structure, a large number of mineral particles would be obtained free from all voids or air spaces, and the average specific gravity of all these particles would then represent the true specific gravity of the substance composing the original mass.

The converse is true regarding a determination of the true specific gravity of a material. In other words, as the size of particles is decreased it should be easier to determine the true specific gravity than where the original structure is more nearly preserved. It may be argued that in any displacement method, by allowing a specimen to remain in water until absorption is complete, the voids will be filled, and if in some manner the water filling these voids is measured, a correction may be obtained which will make possible the determination of the true specific gravity of the material. It is probable, however, that this is not the case, as the absorption of a large fragment may be relatively less than that of a small fragment, owing to the fact that water first absorbed from the outside may imprison air which is present within the structure and prevent its escape. From a mechanical standpoint, however, there is a limit in connection with the fineness of particles best suited to obtaining true specific gravity, namely, the fact that a mass of very fine particles is more apt to entangle air, which cannot be removed by agitation, than is a mass of larger particles. By any displacement method, therefore, the entangled air would vitiate the determination to some extent.

In accurately determining the amount of absorption of a mass of mineral matter where the absorption is weighed, there must also be a certain limit to the size of particles used, for it is necessary to surface-dry these particles after the absorption is complete before weighing them, and any error resulting from imperfect drying must be relatively greater for a mass of very fine particles than

---

*Abstract from paper read before the American Society for Testing Materials.

for the same weight of larger particles, owing to the tremendously greater surface area exposed. Moreover, if surface-drying itself is perfect, where a great number of fine particles are surface-dried the evaporation of the water absorbed by some particles is likely to proceed while the remainder are being dried.

Where extremely accurate determinations are not required, some of the factors discussed above become negligible as applied to certain types of materials, but in other cases they may prove to be quite important. The necessary degree of accuracy of a determination of the specific gravity is a matter of question, but as such determinations are seldom reported to less than the third significant figure, a single unit of which, applied to road-building materials, may represent more than 0.333 per cent., it is presumed that necessity for determinations accurate to this point are usually desired. This is certainly the case in connection with the use of the specific-gravity determination in ascertaining the voids in pavements constructed of rock, gravel or sand, bound together with a bituminous material.

Conclusions.—As the results of the investigations above described, the following conclusions have been drawn:

1. In the case of rock and slag an appreciable variation may exist between apparent specific gravity and true specific gravity, depending upon the absorption of the material.

2. The displacement method formerly used by the Office of Public Roads and Rural Engineering is not as satisfactory as the Chapman method for determining the apparent specific gravity of single specimens of rock, slag or gravel, unless the difference between apparent specific gravity and true specific gravity is negligible.

3. The Chapman method is satisfactory for determining the apparent specific gravity of single specimens of rock, slag, etc., but is not a safe one to employ in determining the specific gravity of non-homogeneous or apparently homogeneous aggregates, even when the average results of three apparently representative specimens are taken.

4. In the case of non-homogeneous aggregates consisting of fragments of not less than ½ in. in diameter, either the Goldbeck method, the Chapman wire-basket method or the Hubbard-Jackson method are satisfactory, and can ordinarily be depended upon to give check results by different operators working upon the same sample to within 1 in the third significant figure.

5. It is impracticable by any of the methods studied to determine the apparent specific gravity of samples composed of fragments smaller than ½ in.

6. When determining the specific gravity of extremely non-homogeneous aggregates it is recommended that an average of not less than three tests made upon different samples by the Goldbeck, Hubbard-Jackson or Chapman wire-basket method be reported.

7. It has been found that the Bureau of Standards' modification of the Le Chatelier apparatus is more convenient and rapid for determining the specific gravity of aggregates, the individual fragments of which are less than ¼ in. in diameter, than is the Jackson apparatus.

8. When it is desired to obtain as nearly as possible the apparent specific gravity of aggregates consisting of a mixture of coarse and fine particles, it would appear advisable to separate a weighed sample of the material by means of a ½-in. screen and make an apparent-specific-gravity determination upon not less than 1,000 g. of the coarse fraction, and a true-specific-gravity determination upon not less than 50 g. of the finer fraction. The specific gravity of the whole may then be calculated from the following formula, where W equals the percentage by weight of coarse aggregate, W' the percentage by weight of fine aggregate, and G and G' their respective specific gravities:

$$\text{Specific Gravity of Aggregate} = \frac{100}{W/G + W'/G'}.$$

VIBRATION IN STEAM TURBINES.*

By H. A. Fisher.

BEFORE the advent of the steam turbine in the modern power station, vibration was not a serious consideration, in fact was practically unknown. In the reciprocating engine with its low speed any lost motion or misalignment made itself known as a knock or pound, while in the steam turbine with its high speed and delicate balancing the knock and pound becomes vibration. It is the purpose of this article to set forth some of the cases of vibration that are commonly observed.

Revolving parts improperly coming in contact with the stationary parts will cause vibration. While this is not a frequent occurrence, it is one that will do the most damage and therefore, when suspected, no time should be lost in making an examination to see if there is such contact and if so to remedy it at once.

Faulty grouting under the bedplate is another dangerous source of vibration. The proper mixture and mixing of the grout is important, for no matter how much more care is exercised in placing the grout, it will not be satisfactory if it has not been thoroughly mixed in the right proportions; a mixture that is too rich (contains too much cement) will shrink a great deal in setting, while one that is too lean (contains too much sand) will crumble. A grout of equal parts of cement and sand, thoroughly mixed dry and wet down to the consistency of a thin batter and promptly poured, will give good results when care is taken to eliminate the following causes of poor grouting: Freezing, drying out too fast, too little time allowed for the grout to set before starting the machine, grout becoming soaked with oil which will cause it to crumble, and failure to provide for the escape of air from under the bedplate when the grout is being poured. Vibration due to faulty grouting and the movement of the bedplate on the foundation can usually be felt throughout the entire turbine and remains constant at all loads. This vibration can sometimes be stopped or considerably reduced by driving iron wedges under the edge of the bedplate at several points, taking care not to drive them too hard, as the bedplate may be sprung and the alignment of the machine disturbed. The only permanent remedy is to remove the old grout, level the machine up and grout it in again. The top of the foundation should be made rough so that the grout may bind and the foundation should be thoroughly wet down just before the grout is poured.

Turbines supported on I-beams or set on floors supported by I-beams should be grouted with lead, as this will absorb the vibration and prevent its transmission to other machinery and to the building. The final leveling of the bedplate should be done after the turbine and generator have been placed on it and secured.

---

*From "Power" of New York.

Bearings that have too much journal clearance—that is, too much clearance between the shaft and the bearing —cause vibration. Such vibration is of a more or less local nature; while it may be felt throughout the machine, it is more noticeable at the bearing or bearings that are loose and is usually more noticeable when the machine is running with a light load and in alternators when the field current is off, becoming less as the load increases or the field magnetism becomes stronger holding the rotor in a more fixed position. Feeding more oil to the bearings will sometimes reduce the vibration.

Journal clearance should be measured with a feeler gauge or determined by pinching a lead wire between the top half of the bearing and the shaft and afterward measuring its thickness with a micrometer. This clearance should be about 0.008 in., for a few thousandths more will sometimes cause vibration. In case the journal clearance is found to be correct and the vibration continues, the bearing shell should be examined to see if the lining is loose or the bearing is not rigidly held in the pedestal or pocket in which it rests, as any clearance between the bearing cap and shell will allow vibration, which will gradually become worse.

Clearance between the bearing cap and shell is sometimes caused by inaccurate fitting in the shop, or by some foreign material between the machined faces or flanges of the bearing cap and pedestal preventing the cap from being drawn down on the bearing shell, or by the practice of putting a heavy coating of thick shellac or other substance on this joint to prevent leakage of oil. In one case the writer found that a paper gasket had been put in, causing a clearance of 0.006 in. between the cap and the bearing. Ascertaining the clearance between the cap and bearing shell is accomplished in the same way as journal clearance. The bearing should be placed in its regular position and several pieces of small lead or fuse wire placed on the machined surface of the bearing shell so that when the bearing cap is placed in position and the bolts tightened, the soft wire will be squeezed down to the thickness of the clearance between the bearing cap and the shell. It is best to start with a small wire and if no impression is made on it, try a larger one. After getting an impression of the clearance, it should be measured with micrometer calipers, and a thin brass or copper shim of the same thickness should be put in. If the shaft is low at the bearing the shim should be placed under the bottom half of the bearing shell, but if the shaft is level the shim should be placed on top between the shell and cap.

With high-speed turbines it is necessary that the rotor be perfectly balanced. This is done in the shop, but during shipment or in course of erection, or even after the turbine has been put in operation, the shaft may be sprung, throwing the machine out of balance. If it is not sprung more than, say, 0.01 in., the rotor may be balanced by weights in the drum or disks of the turbine, but this should be done by someone who has had considerable experience in this kind of work, for weights in the wrong place may do considerable damage. Turbine rotors may be thrown out of balance by a leaky throttle allowing steam to enter when the turbine is standing still. This heats the rotor and the casing unevenly, and the rotor will run out of true and vibrate when starting. The machine should not be left standing with the steam blowing through, and it is best to start it rotating slowly while warming up.

Scale and mud in the blades or buckets will also unbalance it and should be removed as soon as possible. Dirt on the rotor of the generator will sometimes cause vibration, therefore the generator field and armature should be blown and cleaned frequently.

The coupling between the turbine and generator should be examined for loose bolts, and in machines having flexible couplings the fingers or pins of the coupling should be examined for wear, as a shoulder worn on the pins may cause the shaft to have endwise travel and there will be a sort of bumping vibration that will be more apparent at light loads. In this type of coupling the oilways should be kept open so that the oil may get between the pins.

The generator air gap should be equal, and the rotor and stator should be axially centered with each other so that there will be no end thrust. Loose bearings on the governor or oil-pump shaft may cause vibration. The steam and exhaust piping should be provided with expansion joints and be well supported, so as not to impose strains on the turbine and disturb its alignment. The running clearances being small, care should be exercised when the bearings are disturbed or adjusted in any way, so that the proper clearance is maintained between the stationary and revolving parts.

In conclusion, there is nothing mysterious about the steam turbine or its ailments, so that any of the ordinary ailments and adjustments are well within the scope of the engineer.

TO MAKE STEEL BY ELECTRIC REFINING.

"Norway and Sweden now have an output of over 100,000 tons of iron and steel produced by electro-metallurgy, but it is claimed that every condition existing in those northern countries which makes such manufacture possible and profitable is to be found in British Columbia.

"This Scandinavian steel is smelted by electricity, demanding cheap power, and the electric current can be best obtained where there is unlimited water power. British Columbia is so abnormally rich in falls well situated for commercial purposes, that it would appear as though electro-metallurgy must become eventually a most important industry. The process now developed in Sweden is as follows: The molten pig is delivered direct to the open-hearth furnace, heated by gases from the reduction furnace, and finally the charge from the open-hearth furnace is poured into an electric refining furnace, and the steel is of remarkable high quality. This is obviously the right way, if one can command current at a low price."—Exchange.

The Swiss Government decided to favor the single-phase system for electrifying its railways in preference to the third rail, because it costs 10 per cent. less to make the change.

Existing hydro-electric and steam generation power plants serving Vancouver, New Westminster and the entire Lower Fraser valley area in British Columbia, amount to 130,000 h.p. There are water-power sites close enough to Vancouver to be utilized if industrial development demanded it, sufficient to generate 750,000 h.p.

An estimate of 5,100,000 tons of iron ore obtained from quarries in 1915 brings the total iron ore mined in Great Britain during that year to 12,976,105 gross tons, which includes the 6,080,218 tons mined under the Coal Mines Act and the 1,795,887 tons under the Metalliferous Mines Act. The total in 1914 was 14,867,582 tons, and in 1913 it was 15,997,328 tons.

A project for the construction of a tunnel under the Bosphorous is being discussed at Constantinople. Engineers have worked out detail plans showing that the project is feasible and financially profitable, while its strategic importance is obvious. Earlier projects for direct communication between the European and Asiatic shores of Turkey, inspired by the construction of the Anatolian and Bagdad Railway, were devoted to the subject of bridging the narrows at Rumili Hissar.

DESIGNING AN EARTH DAM HAVING A GRAVEL FOUNDATION, WITH THE RESULTS OBTAINED IN TESTS ON A MODEL.†

By Messrs. E. C. La Rue, George M. Bacon, H. A. Petterson, and D. C. Henny.

E. C. LA RUE, Assoc.M.Am.Soc.C.E. (by letter).—This paper will be of exceptional interest, for only meagre data are available relating to the proper design of an earth dam having a gravel foundation. The partly completed dam, described by Mr. Hays, formed a part of an irrigation project on which the writer has prepared a water supply report. He has also examined the dam site, and is therefore specially interested in the paper.

In some parts of it the author has not made clear the procedure followed in his experimental work. For example, in determining the hydraulic gradient of the material used in the original dam and that to be used in the upper portion of the actual structure, the details of the methods are not made clear. The writer is in doubt as to the following points:—

1. When the readings were taken, was the quantity of water leaving the same as that entering the tank?

2. Before the soil was placed in the tank, were tests made in order to determine the head required to force the water through the valve? If the coarse materials were tested to determine the hydraulic gradient, then, with the same quantity of water flowing into the empty tank as that used in the test, the head required to force this water through the valve could be determined. If, during such tests, it was found that the water level in the tank was above the intake to the lower glass tube, then the author's results are in error. During the tests, with the soil in the tank, the head required to force the water through the valve would be effective to a higher level in the tank. It is probable that the water level in the lower tube was not affected in the manner referred to, but the author has not made this point clear.

On page 323,* in referring to the North Dike of the Wachusett Dam, Mr. Hays states:—

"A flat hydraulic gradient, caused by the water in the reservoir seeping under the dam, called for a large quantity of material, in order to withstand the upward pressure under the down-stream portion of the dam."

In referring to the Gatun Dam, he states:—

"A flat down-stream slope causes the percolating water to travel a long distance before a free opening is encountered, thus causing the upward pressure to be consumed by friction."

As the down-stream sections of these dams are constructed of pervious material, the water cannot exert an upward pressure on their bases. Probably the author means that the flat hydraulic gradient makes it necessary to construct the down-stream section of the dam with a flat slope in order to prevent the line of saturation from intersecting the down-stream face of the dam above the toe.

On page 333,* in referring to the hydraulic gradient for the selected material to be used in the upper section of the actual structure, the author says: "From this it was assumed that the hydraulic gradient was not greater than 1 : 1, although it was evidently much steeper." This statement is not consistent. It would appear that he meant to say that the hydraulic gradient was not less than 1 : 1, and it was evidently much steeper.

On page 333,* in referring to the model dam, the author says:—

"The long up-stream slope was given in order to allow the downward pressure of the water over the up-stream section to have a balancing effect on the upward pressure beneath the dam, as blow-outs would be improbable in this portion of the dam."

Although the down-stream pressure of the water over the up-stream section does have a balancing effect on the upward pressure beneath the dam, this surely was not the reason for adopting a "long up-stream slope." The author assumes that the material in the up-stream section will be practically water-tight. It would appear, therefore, that the flat up-stream slope was adopted in order to force the hydraulic gradient to begin farther up stream.

The author concludes that, with 10 in. of water in the reservoir above the model dam, the head is entirely consumed before the water reaches the lower toe of the dam. Using Test No. 17, Table 3, the writer has drawn lines of equal pressure, and from these produced the lines representing approximately the direction of flow through the foundation of the model dam. The direction of flow was downward in that portion of the foundation above the sheet-piling. Below the sheet-piling the flow lines rose slightly and then appeared to turn in the direction of the drain valve, shown in Fig. 8. If the pressure had been observed at various points in the foundation below the lower toe, there is no doubt that it would have shown conclusively that the direction of flow was toward the drain valve. Mr. Hays has shown the hydraulic gradient for the material used in the foundation to be about 1 : 9. With a free escape for the water, as is provided by the drain, little, if any, pressure would be observed immediately above the drain valve. Extending the hydraulic gradient back from the drain valve with a slope of 1 : 9, it will be seen to intersect the line representing the base of the dam at a point 2,160 ft. above the drain, or 1,040 ft. up stream from the upper toe of the actual dam. All the pressure observations taken by the author must have been affected by the drain. Under these conditions, the water could not rise to the base of the dam near the lower toe. In the model, the author has imposed conditions which would not be reproduced in actual practice. If the dam is to operate under the same conditions as the model, then 360 ft. below the lower toe of the dam a trench, 240 ft. deep and 40 ft. wide, must be constructed across the canyon. This trench must be filled with large stone, in order to provide a perfect drain. In the bottom of the trench, or at the 240-ft. level, there must be openings with sufficient capacity to carry off the water as it arrives. The water must then be conveyed to a reservoir of infinite capacity. In the model, the drain should be 216 in. from the lower toe, instead of 36 in., or it will affect the water pressure under the base of the dam. If the writer's contentions are correct, the results of the author's experiments are of little practical value.

That the drain referred to has affected seriously the results of the experiments is indicated by the excessive seepage through the foundation of the model. In Test No. 17, Table 3, the seepage, per linear foot of the model, was 0.00222 cu. ft. per sec. The slope of the line of saturation in the model is assumed to be the same as that of the actual dam. The length of travel in the model is

†Discussions of a paper by Jas. B. Hays as published in Proceedings of American Society of Civil Engineers for August, 1916.

*Proceedings, Am.Soc.C.E., for March, 1916.

*Proceedings, Am.Soc.C.E., for March, 1916.

proportional to that of the actual dam. The head and all other dimensions in the model being proportional to those of the actual dam, the seepage through the latter, per linear foot, will be 120 times that through the model per linear foot. The length of the actual dam is to be 2,000 ft., and its left end will extend into a bank of gravel similar to that which is to compose the foundation. Considerable excavation will be necessary in order that the water-tight section of the dam can be carried well into this bank. It is assumed that the area exposed to water pressure will be equivalent to the area of 1,200 ft. of maximum section. The seepage through the actual dam, therefore, would be $120 \times 1{,}200 \times 0.00222$, which equals 320 cu. ft. per. sec.

Mr. Hays has not disproved the "line of creep" theory. In the first test, with the two rows of sheet-piling, he found a small loss of head at the upper row of piling. This, no doubt, was due to water passing through the upper section of the dam and entering the foundation both above and below the upper row of sheet-piling.

The pressure at *D*, Fig. 8, below the lower row of sheet-piling, was no doubt due to the water from the upper section of the dam entering the foundation between the piling and the cut-off wall. In fact, the drain would prevent the water from rising to *D* after passing below the lower row of sheet-piling. If Mr. Hays will place the drain 216 in. below the lower toe of the model dam, separate the upper section of the dam from the foundation, with a sheet of tin, and connect the tin with the cut-off wall and the two rows of sheet-piling with water-tight joints, then the hydraulic gradient will begin at the upper toe. Under these conditions, the effect of the two rows of sheet-piling can be determined, and undoubtedly the "line of creep" theory, somewhat modified, will prove to be correct. That is, instead of following down one side of the sheet-piling and up the other, thence along the base of the dam to the second row of sheet-piling, etc., the water will follow down the upper side of the first row of sheet-piling and thence in the general direction of the lower end of the second row of sheet-piling.

Assuming that there were no other defects in the model, the writer believes the results of the tests to be unreliable for the following reasons:—

1. The model dam, being only 11 in. high, and subjected to a head of 10 in. of water, the entrance head and capillary action would, no doubt, affect the hydraulic gradient to such an extent that the pressure observations in the model would not indicate the action of the water in the final structure, where similar material will be subjected to a head 120 times greater than that on the model.

2. The model dam was constructed of the same material as that to be used in the actual structure. It would seem that the material for the former should have been coarser than that to be used in the latter. The ratio between the coarseness of the materials in the two structures, which would result in the action of the water on the model being comparable to that on the actual dam, could perhaps be determined by extensive experiments.

The writer makes the following suggestions:—

1. That the author construct a model which will represent exactly the original dam as it was to have been built. It is probable that the results of tests on this model would show the structure to be safe, provided the model were constructed with the drain 36 in. below the lower toe. As a matter of fact, with a head of 24 ft. on the actual dam, the water passed freely through the partly completed structure.

2. That the author could obtain more reliable information from a series of tests on models 1, 2, 4, 8 and 12 ft. in height, provided the drain shown in Fig. 8 were placed at a section 216 in. down stream from the lower toes of the respective dams. From the results of the experiments with each of these models, it is possible that some sort of a curve could be prepared which might be extended to show approximately the action of the water on the final structure under a head of 100 ft.

Conclusions.—1. The author has not stated clearly how each experiment was carried on. In some places it is necessary for the reader to assume that the experiment was conducted under certain imposed conditions.

2. The model dam used by the author was too small to give reliable results.

3. The pressure, at all points observed in the tests, was affected by the drain; therefore, the results of the experiments are of little practical value.

4. The drain in the model should have been placed 216 in. below the lower toe, instead of 36 in.

5. The author has not disproved the "line of creep" theory.

6. It would appear that too much dependence was placed on the impermeability of the material in the upper section of the dam.

7. If the dam, as designed, were constructed to operate under the conditions imposed in the model, the flow under it would be about 320 cu. ft. per sec.

8. It is the writer's opinion that the sheet-piling should be placed beneath the core-wall. If the core-wall in the partly completed dam is to be used, then the sheet-piling should be placed near the core-wall on the up-stream side. A water-tight connection should be made between the core-wall and the sheet-piling. The writer, being somewhat familiar with the conditions at the dam site, feels that, at best, there will be considerable seepage under the dam, and for this reason he would suggest that the slope of the down-stream face, below the 30-ft. berm, be made as flat as 3½ : 1 or 4 : 1. The proper slope for the down-stream face could no doubt be determined by extensive tests on a series of properly designed models of various heights.

George M. Bacon, M.Am.Soc.C.E. (by letter).—The main objection to drawing conclusions from these experiments seems to be the assumption that the foundation as composed for the model represents sub-surface conditions at the dam site, an assumption hardly correct. What part of the foundation in the model corresponds to the actual condition on the ground, which allowed sheet-piling to sink "as deep as 32 ft. with one or two blows from a 1,700-lb. hammer"? The mountain streams formed "great cones, or fans, of very porous material." Was this material duplicated in the model, and, if so, where? In an experiment of this kind, it is vital to duplicate the actual conditions which are the subject of investigation. No ingenuity of observation and recording can minimize the importance of this. There is practically nothing in the paper showing how the foundation in the model was formed, or indicating its similarity to actual conditions.

The author's theoretical analyses are interesting, but, should they serve as a basis for the solution of the problem actually presented? If the premises are not correct, any deductions from experiment are not only of no value, but can easily be harmful as well as misleading.

H. A. Petterson, Assoc.M.Am.Soc.C.E. (by letter).—The writer is a great believer in experimental engineering, and realizes that much of the advance made in engineering knowledge is due to the researches of careful and ingenious experimenters. He cannot believe, however, that experiments made on models with a depth of water of only 10 in., will bring forth results of any great practical value in designing a dam to impound water 100 ft. and more in depth. Our knowledge of the underground flow of water is not as complete as it ought to be. This is true especially of underground flow as affected by cut-off walls penetrating only part way into the porous stratum.

The most reliable experiments, however, would be those made on existing dams and weirs built on porous foundations. There are many structures in different parts of the world on which experiments could be made, and these could follow essentially the methods developed by Mr. C. S. Slichter. Until such experiments are made, the writer, for one, would advocate following present methods, a brief presentation of the underlying principles of which will be given.

The principles governing the design of an earth dam with impervious core-wall to impervious foundation need not be reviewed, as they are treated in any number of good textbooks. The problem of securing water-tightness in earth dams is essentially one of securing the maximum density of the material; and the laws governing this are known by the engineering profession, even though not universally applied. Strict adherence to these principles in earth dam construction involves extra cost, which is not always warranted by the results obtained.

This discussion, therefore, will be confined to the principles involved in the design of an earth dam on a porous foundation of such great depth that an impervious cut-off wall to an impervious stratum is financially impracticable. It will be assumed that the dam will be made relatively impervious, and safe against ordinary methods of failure. The principles to be elucidated are the securing of stability against the possible destructive effect of water flowing under the dam (not through it); also, as it is assumed that the dam is to impound water in a reservoir, the investigation of the quantity of water lost by percolation is important from an economic standpoint, though it may in no way affect the stability of the dam.

A rational design cannot be made without a comprehensive grasp of the laws governing the flow of underground water. A very brief summary of present knowledge on these laws will be given, for the purpose of calling attention to the incompleteness of that knowledge and to make clearer the writer's comments on certain of the author's statements.

Laws of Underground Flow.—Hazen's formula, reduced to English units, is

$$Q = 3.28\, c\, d^2 \frac{h\,A}{L} \quad \frac{(t + 10)}{60} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Slichter's formula is

$$Q = \frac{16{,}272\, d^2}{k} \quad \frac{h\,A}{L} \quad [1 + 0.0187\,(t - 32)] \quad \ldots\ldots (2)$$

Baldwin-Wiseman's formula is

$$v = c_1 c_2 \frac{h}{L} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

In these formulas, c, c_1, c_2, and k, are coefficients. The values of c in Hazen's formula vary from 400 to 1,000. The values of k are tabulated by Slichter, and vary only with porosity. c_1, in Equation (3), is proportional to cd^2 and $\frac{d^2}{k}$ in Equations (1) and (2). c_2 corresponds with the temperature correction of Equations (1) and (2).

Q = discharge, in cubic feet per day, through the area, A;

A = area of cross-section, in square feet, normal to the line of flow;

h = difference of water surface, in feet, for two points distant L feet apart;

L = distance, in feet, measured in direction of line of flow;

d = effective size of sand grains, in millimeters, determined by mechanical analysis with sieves in Hazen's formula, and by the use of King's aspirator in Slichter's formula;

v = velocity of percolation;

t = temperature of the water, in degrees, Fahrenheit.

These three equations all agree in several respects:

First.—The rate of flow increases with temperature.

Second.—The rate of flow increases with the first power of $\frac{h}{L}$, or, if h is constant, varies inversely with L.

Third,—The rate of flow varies with some power of the effective size of the sand grains.

By platting on logarithmic paper, a straight-line relation will be found to exist between Slichter's values of k and the porosity of the material. The same relation holds between porosity and the tabulated values of the transmission coefficient in Water Supply Paper No. 140. It may be shown that Slichter's tabulated results may be expressed in the following form:

$$Q = \alpha\, p^{3.3}\, d^2 \frac{h}{L} A\, T \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

where p = porosity and T = temperature correction.

The effect of porosity is taken into account in all the equations. In Equation (1), the values of c vary with the uniformity coefficient, which is an indirect and approximate method of stating the effect of porosity. In Equation (3), the effect on porosity is introduced in the coefficient, c_1.

The writer believes that further experiments are required before a formula for underground flow will be developed, which will be even approximately as accurate, for instance, as that for the flow of water in pipes and other conduits. Based on present knowledge, such a formula will have the general form:

$$Q = \alpha\, p^n\, d^m \left(\frac{h}{L}\right)^r A\, T \quad \ldots\ldots\ldots\ldots (5)$$

Our present formulas are applicable to sands up to an effective size of 5 mm. Hazen specifically limits the range of his formula to sands of from 0.10 to 3 mm. effective size. The writer has introduced the exponent, r, in Equation (5) as necessary if the formula is to be applicable to all sizes of material. Thus, for large boulders, there is no doubt that r would approach a value of 0.5, as for flow in channels; and, for extremely fine material, such as silt and clay, r may become greater than unity.

Considering the equations for underground flow, we may, for convenience, combine the factors denoting effect of porosity, effective size, and temperature, and write an equation:

$$Q = \frac{K\,h\,A}{L}, \text{ or } p = \frac{K\,h}{L} \text{ for } A = \text{unity}.$$

The coefficient, K, may be taken as a constant for any given case, as for the gravel stratum under the dam described by the author. The head, h, is also fixed by the storage requirements. The only variables, then, are q and L. The hydraulic gradient equals $\frac{h}{L}$ and may have an infinite number of values, depending on the variable, L. There is, however, a definite relation between K and the character of the underground material.

The writer fails to grasp the author's meaning, when he refers to the hydraulic gradient of various gravels, as though there was only one possible hydraulic gradient for a given gravel. Thus, on page 325,* the author states:

"Having determined the hydraulic gradient of the underground material, a trial design was made to find what dimensions would be necessary in a dam constructed wholly of this gravel."

Again, on page 326*: "This combined material was then tested in the tank to determine the hydraulic gradient." On page 333,* there is a reference in similar vein.

Application of Formula for Underground Flow to Design of Dam.—The underground flow may endanger the dam in two ways: (1) The velocity of flow may be sufficient to carry away with it the finer materials in the gravel, and the so-called "piping" action results. If this keeps on indefinitely, it is only a question of time until the underground stratum becomes so porous that all the water in the reservoir will readily escape, or the stratum will settle, and the superstructure will go down with it. (2) The upward water pressure under the base of the dam may be great enough to overcome the weight of the super-incumbent earth, and so-called "blow-outs" occur.

(1) Stability Against Piping.—Considering the equation, $q = \frac{K\,h}{L}$, it is obvious, from what has been already stated, that q, or unit discharge, can only be reduced by increasing L. As the velocity is directly proportional to q, it, too, can only be decreased by increasing L, or, what amounts to the same thing, by increasing $\frac{L}{h}$.

These coefficients are deduced from observations of existing structures, largely in India. They are not the results of experiments, but of compilations of data on structures which have proved to be safe. The writer cannot understand, then, why there is any lack of precedent in designing dams on porous sand foundations. The Laguna Weir is a notable example of a diversion dam founded on extremely fine silt. He agrees with the author as to the value, and even need, of further experiments, if this branch of engineering is to have a scientific basis, but cannot agree as to the value of experiments on models carrying only 10 in. of water.

A particular need for further investigation is the definite determination of the influence of cut-off walls, penetrating only part way through the porous stratum, in reducing the pressure head. According to the "line of creep" theory, this effect depends only on the depth of the barrier and the length of base, L; or, if L_c represents the depth of penetration of this cut-off wall, and h_s the reduction of the pressure head, then

$$h_s = \frac{2\,L_s}{\frac{L}{h}}.$$

The writer believes that the value of h_s depends, not only on L_s and $\frac{L}{h}$, but also on the ratio of L_s to the total depth of the porous stratum. If S represents the depth of the porous stratum, and other symbols are as before, then $h_s = \frac{B\,h}{L}\left(\frac{L_s}{S}\right)^n$ is the form of expression which, the writer believes, presents the value of the lost head. B is a coefficient, and n an exponent.

A point of practical importance is the difficulty encountered in making these cut-off walls impervious. Sheet-piling is assumed by some engineers to be practically impervious, but experienced men point out the tendency for deep piles to spread, and mention the difficulty of preventing this tendency, even with interlocking steel piles. As the work cannot be inspected, there is always uncertainty as to the efficiency of sheet-piling.

(2) Stability Against Upward Pressure Under the Base of Dam.—The remedy against this method of failure is obvious. If y = the pressure head at any point, w = the weight per cubic foot of water, and m = the weight per cubic foot of the material in the dam; then wy is the intensity of pressure which is resisted by the weight of superincumbent earth. If z = the height of the dam at any point corresponding to y, then, for safety, mz should be greater than wy.

The foregoing discussion assumes that the material in the dam is impervious. If it is porous, the water plane will rise in the dam to heights corresponding to y. It is necessary, then, that z be greater than y. It is also necessary, of course, that the batter of the down-stream face of the dam be sufficiently flat to prevent sloughing of saturated material. The batter of the down-stream face will generally be sufficiently flat to ensure stability against sloughing, if it is designed in accordance with the requirements mentioned under "Stability Against Piping."

The writer believes that a more logical distribution of material is to make a flat down-stream face, with the up-stream face given the usual batter of earth dams, say, 1 vertical to 3 horizontal. The material in the up-stream portion should be the most impervious. The reasons for this are well brought out by George L. Dillman, M.Am. Soc.C.E. Such great care need not be taken in the construction of the lower part of the dam, though the material should be considerably more resistant to flow than that of the underlying gravel stratum. In other words, the engineer should see to it that the dam is constructed so that greater resistance to flow is offered at all points by the dam than by the underlying gravel, but the most impervious material should be on the up-stream side.

The author has reversed the usual method of distribution, in that he makes a steep down-stream face and extends the base up stream. The writer has prepared Figs. 14 to 17 in order to compare the usual design with that of the author. The total width of base in the author's design, as closely as can be determined from his Fig. 9, = 760 ft., and L_s, the depth of vertical barrier, = 125 ft. Then, by the usual theory, L = 760 + $2L_s$ = 1,010 ft. A value of L = 1,000 corresponds to a value of $\frac{L}{h}$ = 10, which would be about the value given, in the tables previously referred to, for the gravel described by the author.

Figs. 14 to 17 show four designs with a value of L = 1,000. Fig. 14 shows a section without any vertical barrier constructed in the gravel. The value of m is taken at 100 and that of w at 62.5. The value of z is computed so that mz = 1.6 (wy) or z = y. This gives a factor of safety of 1.6 against failure by uplifting, where the material of the dam is relatively impervious.

Fig. 15 shows the design when an impervious cut-off wall, penetrating 125 ft. into the porous gravel stratum, is used. The effect of this is to increase L, making it = 2 × 125, or 250 ft. Figs. 16 and 17 are for the same general conditions and assumptions, but they have a flat up-stream slope in accordance with the author's idea. The relative quantities of material per unit of length of dam are given in Table 4. The quantities include only

*Proceedings, Am.Soc.C.E., for March, 1916.

those portions of the dams above the natural ground surface. The sections in Figs. 14 and 16 are directly comparable with those for Figs. 15 and 17. Attention may be called to several things in Table 4.

First, as far as total quantities are concerned, the section proposed by the writer has no material advantage over that proposed by the author. The quantity listed as up-stream material is far less, however, for the dams shown by Figs. 14 and 15 than for those shown by Figs. 16 and 17. In view of the foregoing, the dams with the long down-stream slope can be constructed more cheaply.

Table 4.—Volumes, in Cubic Yards per Linear Foot, for Dams Shown by Figs. 14 to 17, Inclusive.

Up stream.	Down stream.	Totals.
754	876	1,630
754	524	1,278
1,242	448	1,690
841	448	1,289

Second, the dam with the long down-stream slope is likely to be much more efficient in reducing the rate of percolation under the dam. Unless extremely impervious

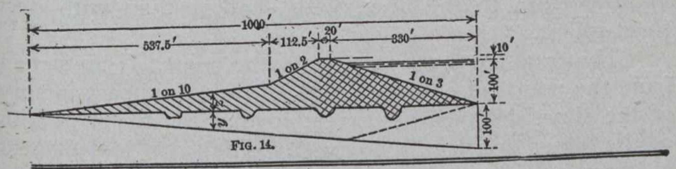


Fig. 14.

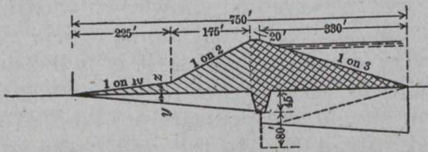


Fig. 15.

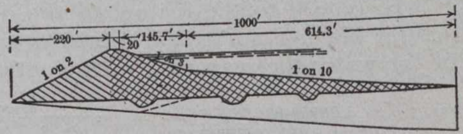


Fig. 16.

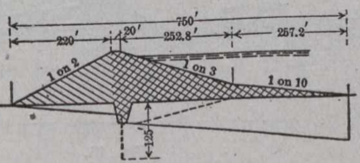


Fig. 17.

material is secured for the shallow portion of the up-stream section of dams like those of Figs. 16 and 17, it is very probable that there would be percolation downward through the dam, near the heel, adding to the underground flow. In other words, the value of $\frac{L}{h}$ for a dam of this type is less than for a dam of the type shown by Fig. 14, even with the same length of base and the same depth of sheet-piling, or other cut-off wall.

D. C. Henny, M.Am.Soc.C.E. (by letter).—The design of a dam in a situation such as the author describes presents an interesting and difficult problem. In his conclusions he mentions several causes of earth dam failures, and states that his paper deals with one cause only, namely, springs or boils, which might produce piping under the base of the dam.

The usual percolating velocities are exceedingly low, far below the power of transporting material. The piping or blow-up phenomenon implies a combination of circumstances, relating to the material in place, essentially differing at particular points from the ordinary percolating conditions. In the case of clayey foundation, initial cracks may transmit a large portion of the available pressure to a point close to free exit, and may set up progressive erosion aided by arching. In the case of sand and gravel, strata of unusual openness to flow may be contiguous to layers of very fine sand. In very coarse material, the open spaces between pebbles and cobbles may be so great as to preclude true percolation and permit comparatively free flow and high velocity. The latter case is one which cannot be regarded as applying to foundations for high dams. The other cases are dependent on local deviations from general homogeneity, which for large areas can hardly ever be known definitely. Moreover, the effect of such deviations cannot be ascertained by experiments with selected samples of materials placed in tanks or boxes.

The experiments conducted by the author do not appear to have had for their object the determination of a maximum gradient which would be safe against piping, but rather the most economical form of dam which would produce a maximum reduction of the water gradient, thereby minimizing the piping danger as well as seepage losses.

It appears to the writer that, whatever may be the danger from piping, it must be judged by examination of test pits and experience with existing dams. Usually, with the foundation material which the author describes, and with ordinary slopes of an earth dam, such danger is not great and, if necessary, can be counteracted economically by a gravel blanket on the ground below the down-stream toe. Nor need there be any fear of bank sloughing, if gravelly material similar to that in the foundation is used in the down-stream portion of the dam. The real problem seems to be that of insuring against excessive seepage losses such as would render the reservoir useless.

In describing the history of the reservoir, the author states that when the dam was completed to a height of 30 ft., it was subjected to a head of 25 ft., at which time water escaped in considerable quantity from the down-stream toe. Though the quantity of water escaping is not stated, it is evident that if this is known or can be ascertained approximately a full-size experiment is at hand on which to base some judgment as to the seriousness of the problem.

A portion of this escaping water may have come through the dam proper. It is certain, however, that by far most of it passed through the dam foundation, which is described as being of a very porous nature and of unknown depth, roughly estimated at 240 ft.

If, at that stage of completion, the dam had a full width of base of approximately 500 ft., the water gradient producing this heavy seepage may have been 1 to 20, and a statement of the seepage per linear foot of dam would permit some judgment as to the practical admissibility, from the storage point of view, of such gradient for a full reservoir.

The author's determination of a safe gradient of 1 to 9 by vertical tank test is by no means conclusive. The essential feature, namely, the quantity of seepage with such gradient, is not stated. However, even if it were stated and were satisfactorily low, it is necessarily based on the use of samples of foundation material which, in the nature of the case, cannot represent any known sort of average of the deep masses of gravel, sand, cobbles, and boulders as they lie in place under the dam. Homogeneity cannot exist to any degree in material of greatly varying sizes deposited by successive floods of varying intensity.

Assuming, however, that the samples used in the author's tests are representative of the general foundation, the tests made in a rectangular box with a model of a dam with varying depths of tight cut-off show the quantity of seepage under full head. For the first model tested, this averaged approximately 0.0040 sec.-ft., and, for the second model, 0.0025 sec.-ft. per lin. ft. of model. The writer understands that these quantities refer to the flow measured in the experiments, on a scale of 1/120 of full size. If this understanding is correct, then, so far as the experiment goes, the deduction may be made that, for similar material in the foundation, the seepage for a dam built on the basis of the second model would be 120 × 0.0025, or 0.3 sec.-ft. per lin. ft. of dam.

The longitudinal section shows the dam to be 400 ft. in length across the general river bed and 1,600 ft. in length on the adjoining bench. No data are at hand as to rise of rock under this bench. It may be interesting, nevertheless, to inquire as to what the total seepage under the dam would be if the rock were to rise but slightly away from the river. In that case the average gradient and seepage per linear foot for the bench portion of the dam would approximate one-half that for the full height of the dam. On such assumptions, a seepage would result of $\left(400 + \frac{1,600}{2}\right) \times 0.3 = 360$ sec.-ft. This quantity of seepage is clearly inadmissible, and the writer deems it likely that some of the foregoing assumptions may be known by the author to be erroneous.

Independent of the doubt regarding test samples being representative of material in place, there must be serious uncertainty as to the possibility of driving sheet-piling to a depth 80 ft. below the bottom of the cut-off trench, and as to the tightness of such sheet-piling when driven.

It will be noted that the final design of the dam as presented by the author shows an approximate gradient of 1 to 10 for full reservoir. This may be about twice the gradient which prevailed when there was 25 ft. of head against the present dam, at which time heavy seepage losses occurred.

A detailed study of the pressures as registered in the experimental box with the model of the dam reveals some marked inconsistencies, which, if the experimental results are to be made the basis for design, may require explanation. In considering this subject, the writer has confined himself to the use of experiments made under a full head of 10 in. representing 100 ft. on the scale of the experiment, and has selected for this purpose only those numbers, four in the first and three in the second series, for which pressures are recorded at all points.

Individual differences of flow and pressure are rather large, the maximum variations from the average being as follows:

	First series: Experiments 9, 14, 19 and 24.	Second series: Bxperiments 4, 8 and 12.
Flow	7%	20%
Pressure	20%	13%

In order to eliminate individual variations, whatever may be their cause, pressures and pressure drops are figured on the basis of average values and are listed for comparison in Table 5.

Table 5 shows the following rather surprising results as to pressure head destroyed by percolation: The pressure drop is greater from open water to A, from B to C, and from D to E, with smaller than with larger flow; the pressure drop is greater from A to B without sheet-piling, and with small flow than with sheet-piling and with large flow; the pressure drop is greater from C to D, in proportion to the flow, with shallow than with deep sheet-piling.

Table 5.—Pressures and Pressure Drops.

	Average of Experiments.			
	1st Series: Nos 9, 14, 19 and 24.		2nd Series: Nos. 4, 8 and 12.	
Measured flow...	0.0040 sec-ft.		0.025 sec-ft.	
	Pressure.	Pressure drop.	Pressure.	Pressure drop.
Water toe.......	100 ft.		100 ft.	
		58 4 ft.		60.5 ft.
A	41.6		39.5	
Cut-off......	50 ft. in 240 ft.	4.0	25 ft. in 240 ft.	11.8
B	37.6		27.7	
		2.8		5.7
C	34.8		22.0	
Cut-off......	85 ft. in 240 ft.	21.9	125 ft. in 240 ft.	17.7
D	12.9		4.3	
		2 8		3.7
E	10 1		0.6	
		2.2		0.6
F	7.9		0	
	....	92.1	....	100.0

The drop from open water to the point, A, up stream from the points of cut-off, is 60% of the total head, in spite of the short distance of travel; so that the upper portion of the water gradient is steepest. The great loss of head at entrance appears to be inherent in experiments of this kind. It may well be doubted, however, whether such losses occur in the case of actual dams, where the area of entrance is very extensive, unless it is induced by silt deposits. Should no such loss with the actual dam be experienced, and should the pressure at A be 80 or 90% of the total head instead of 40%, the actual seepage losses may be double those deduced from the experiments.

In regard to the feature of the design consisting of a tight blanket up stream intended to lengthen the path of the water the reasoning of the author is believed to be

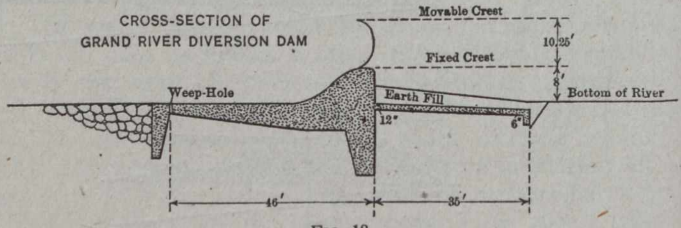


Fig. 18.

sound. The same method was advocated by the writer and was adopted in the case of the Grand River Diversion Dam built by the Reclamation Service near Grand Junction, Colo. The object in this case was the reduction both of uplift and of seepage. Fig. 18 shows a cross-section of this dam.

It may be stated that in this case measurements of uplift pressures were made through pipes placed in piers in the dam and ending in pockets of screened gravel under the foundation. The results indicate complete absence of entrance losses. They also show drop of pressure to be closely proportionate to distance along line of creep. It should be stated, however, that owing to delay in placing movable gates, the heads at the time of the two measurements were in each case only between 4 and 5 ft., and that measurements under a full head of 18 ft. may give different results.

Experiments of the character made by the author must always be of intense interest to hydraulic engineers and deserve full recognition.

CONCRETE TRACK FOUNDATIONS.

AT the last annual Good Roads Congress, held in Pittsburgh, Pa., in February, R. Keith Compton, chairman of the Baltimore Paving Commission, read a paper on "Street Railway Track Construction in Paved Streets." In this paper, after stating the objections raised by electric railway engineers to concrete track foundation, Mr. Compton proceeded to give the experience of Baltimore, which had insisted on 6-inch concrete construction on streets where traffic was heavy. He said in part:—

"In improving streets containing railway tracks the forces of the railway company and that of the paving contractor have to work in conjunction. The railway area is first graded out to the subgrade of the paving by the paving contractor. The railway company then takes charge and grades out to a point six inches below the bottom of the ties. New rails and ties are then installed where necessary, together with any new special work. The ballast, of the size and depth previously noted (1½ to 2½ inches spread to a depth of six inches), is then placed and thoroughly tamped under the ties and up to a point two inches above the bottom of the ties, the rails brought to the proper grade and line, and when the entire construction is 'tight' the penetration begins.

"The grouting mixture is composed of one part cement to two parts sand, and is of about the consistency of thin cream. The operation is readily done without interruption to car traffic by the use of a small continuous mixer (known as the Coltrin mixer) placed just outside of and parallel with the railway tracks, with a flexible chute, in two sections, to convey the grout from the mixer to the ballast. Starting on the down-grade end and working up-grade, the thin grout is penetrated into the stone ballast, which, as previously noted, has already been securely tamped and made to carry the strain of the passing cars. As already noted, the chute is flexible and in two sections. When a car comes along, the first section is thrown out of service and the second section is lowered to the ballast at about the ends of the tires and the mixer kept in service. After the car passes, the first section is thrown back in service.

"It is true that during this operation some movement occasionally occurs in the tracks, but there is a city inspector on the work at all times who hunts for and locates loose ties and they are immediately tamped up with green concrete.

"The natural supposition is that sufficient movement of the ties and track would occur to injure the concrete while setting, but this is not true if the work is carefully handled and executed. On one street in Baltimore this work was successfully handled with five different lines of cars passing up and down the street with but 20 seconds headway at times during the day, while on another piece of work it was successfully installed with eleven different lines of cars passing over the special work with less than 20 seconds headway at short intervals during the day. The resultant mixture is about one of cement, two of sand and five and one-half of stone, with the concrete very dense, as the ballast has been thoroughly tamped and voids reduced to a minimum.

"This ends the work of the railway company, as after this section of concrete is installed the paving contractor again takes charge, installing the concrete base for the pavement immediately on top of the railway base, and then the paving."

Mr. Larned's Discussion.—In the Aera Magazine for August, Mr. J. M. Larned replies to Mr. Compton's paper as follows:—

"Undoubtedly the character of the construction of a street railway track has much to do with the permanency of the pavement within the space occupied. It should be noted, however, that even under the most ideal forms of track construction the pavement in and around the tracks will always be more difficult to construct and maintain than elsewhere, and that some pavements are eminently unfitted for such use. The most permanent and best fitted pavement for the purpose, should be used within the railway area, regardless of the kind of pavement used upon the rest of the street and, as stated by Mr. Compton, block stone pavement, either new or reclipped, having the joints filled with cement grout, will give excellent results.

"In considering the more or less permanent character of the track and of the foundation for the same with respect to the pavement, it will always be necessary to consider the density and weight of the traffic which the track and its pavement is to carry. This, however, should not be the only measure for determining the character of the track structure. A track having light and shallow rails, poor joints and ties spaced over two feet apart, even with a concrete foundation, assuming for the moment only that such foundation is the best, may very well be eminently less permanent and satisfactory than a track constructed with heavy deep rails, the best of joints, having ties spaced not over two feet apart, upon a good natural soil bed or upon a reinforced or crushed rock.

Character of Subsoil Important.—"The first factor which should be considered is the character of the subsoil, its bearing power with respect to the distribution of the load and the necessity for subsoil drainage. It is to be presumed that Mr. Compton in his study of Baltimore conditions took these matters under consideration, but there is no mention made of his having done so, and it would be interesting to know the result of such investigation and study if made.

"It is probably true that the majority of soils under present-day conditions need assistance in the distribution of the load. The distributing factors are the ballast, whether it be of concrete or stone; the ties and their spacing, more so in case of rock ballast though not to be disregarded in case of concrete; the rails themselves, as undoubtedly a nine-inch deep rail weighing 130 to 140 pounds per yard will distribute the load over a much wider space than one seven inches deep, weighing but 108 pounds per yard; and lastly, the joint, which is a very important adjunct, as it is a matter of common knowledge that pavement troubles usually begin at the joint.

"I personally visited Baltimore in 1911 in quest of information about concrete track. I found that at the time the street railway there had recently adopted a heavy nine-inch rail weighing about 132 pounds per yard and were preparing to lay it with an improved joint upon rock ballast and it would appear from Mr. Compton's paper that his investigation has produced instead, a concrete ballasted track with a light rail seven inches deep, weighing 108 pounds per yard, no mention made of the joint, and I fear that this will prove to have been a step backward.

"It is also true that a large majority of street railway officials prefer to use track ballasted with broken stone, rather than with concrete, except in especial cases, and we are certainly interested in the correctness of the solution of this problem, as in most all of our cities, franchise conditions require the railway company at its expense to build and maintain a large proportion of the pavement in streets upon which the tracks are built, and surely there is but little encouragement for the street railway, willing to meet the conditions with a heavily ballasted track of the latest design, when it is confronted with the proposition that upon all heavy traffic streets, concrete ballast shall be used regardless of all other conditions.

Objections to Concrete.—"The objections of street railway officials to a concrete ballasted track are correctly stated in the main by Mr. Compton, though he has failed to state that the work of reconstruction will be materially slowed down and delayed by the use of concrete, except his penetration method of making concrete be adopted. It would certainly be unwise at the present time, after a little over one year's experience upon 10½ miles of track in Baltimore to recognize this penetration method as good practice, in the face of the generally recognized principle that concrete must be allowed to set before being subjected to stress and particularly shock and jar, and particularly also where the cement is floated in by means of grout.

"Grout has its proper uses, but it is not generally regarded as good practice to use grout as the only means of injecting cement into the aggregate. The street railway objections to concrete ballast are real and ever present, and should not be so lightly waived aside. It is a fact that the concrete does in time disintegrate, crack and pulverize under the ties and rails, making a very difficult proposition to handle in making repairs.

"There is much poorly paved track in this country to-day. Such track is either worn-out or has been too lightly constructed, but in many cases the worn-out track has admirably served its purpose and we do not believe that these conditions would have been but temporarily improved had concrete ballast been used; certainly the difficulty and expense of making repairs would have been immeasurably increased and conditions existing in and around track which is manifestly too light and temporary in its nature to meet present-day traffic and pavement requirements should not be a factor in determining conditions governing a relatively permanent track.

Differs with Mr. Compton.—"We cannot agree in the correctness of Mr. Compton's conclusion, after a little over one year's experience in Baltimore, that the concrete-ballasted track is the solution of the pavement troubles in and around street railway tracks. Any properly constructed track, whether ballasted with stone or concrete, should last perfectly for at least ten years without any attention other than that which is due to the vehicular wear on the top surface of the pavement and in connection with this question of life, it would have been interesting and of value if Mr. Compton had included in his paper a list of cities which have used and are using in whole or in part, concrete as a foundation for track. Some information as to the conditions under which it is being used and also a list of cities which have used it and discontinued its use with the whys and wherefores would be interesting. I am not entirely familiar from personal knowledge with conditions at all of the points mentioned, but have made some investigations in the past few years, and it might be proper to call attention to some of these local conditions as follows:—

Experience of Municipalities.—"New Orleans—I have always understood that this city and its streets are literally floating upon soft muck, and that the only way they can get a distribution of the load is by the use of concrete.

"Buffalo—The street railway here was a pioneer in the use of concrete ballast. I visited this city in 1911 and with their roadmaster walked over some of their worn-out track. The ties and rails had completely cut their way down through the concrete and the trackmen were helpless to remedy the difficulty and make repairs. The two types were concrete-beam and solid concrete, similar to the type now being built in Baltimore. These types were both abandoned and since 1912 they have constructed about 40 miles of track, using a concrete slab upon which the track is being laid and surfaced, using, I presume, a little sand for the purpose.

"St. Louis—Has since 1905 and is still using solid concrete track.

"Boston—The street railway has one and one-half miles of solid concrete and 144 miles of concrete-slab tracks and has abandoned these types and is now favoring rock ballast, though the city's requirements still call for concrete slab.

"Brooklyn—Has only ten miles of concrete track, laid in 1907. This type has been abandoned.

"Chicago—The mileage here is about evenly divided between rock ballast and solid concrete ballast. The street railway people say that the rock ballast has proven more satisfactory and that the pavement has held up better. Their tracks in the heavy business portions of the city are largely built upon rock ballast, the concrete is used when the subsoil consists of prairie muck. Some concrete track built upon a good foundation only after three or four years of service developed a crack through the concrete parallel with the rails along the ends of the ties.

"The street railways in the following cities have abandoned the use of solid concrete track after having used the same for a considerable period and having an extensive mileage: Milwaukee, Rochester, Syracuse.

"Philadelphia—With its more than 600 miles of surface tracks has only about 17 miles of concrete construction, the rest is laid generally upon the natural soil. The writer found in visiting this city in 1911 that most of the concrete had been installed upon the heavy-traffic downtown streets, but not upon Market or Chestnut streets, probably the heaviest, where their standard rail weighing 141 pounds per yard has been laid upon the natural soil, the rail having been equipped with a very good joint. This track at that time had been practically worn out in the service and was still in good line and surface and but few if any pavement repairs were needed. During the same visit the writer found at night a large gang at work upon a recently built concrete track, where the cars had been turned off, the pavement removed to permit of shimming up the rails and filling in with grout under them the trough which had been cut in the cement by the rails."

Mr. Campbell's Remarks.—Gordon Campbell, who was the delegate of the American Electric Railway Association to the convention, discussed Mr. Compton's paper extemporaneously, saying:—

"The American Electric Railway Association includes the American Electric Railway Engineering Association, of which the gentleman who has just spoken, Mr. Larned, is a member. This association has a permanent Maintenance of Way Committee, devoting its attention to the study of track questions coming before them and, of course, paving is one of the vital questions. That committee has made a report upon standard track construction laid upon rock ballast and is now studying and working upon the subject of track construction laid in concrete with a view to preparing a report to the association which when adopted will become available to those interested.

"The matter of the construction of tracks in paved streets is obviously one of vital interest to all of us—those who are responsible for the street paving and those who are responsible for the street railway tracks. Since the street railway must furnish its own wearing surface, must supply its own foundation, must also provide a smooth surface for the use of other vehicles, which it does not use, and must maintain this surface, no one will doubt that its engineers are vitally interested, fully as much as any one else, in the permanency of that construction. That the street railways in any great city would fail to look at it from a broad point of view, I should doubt, and I should likewise doubt that engineers, who represent the cities, would entertain any view toward street railways other than those of the greatest fairness and ultimate good to all concerned. What we both seek is cordial co-operation in arriving at the best possible construction for the purpose. It must be obvious to every one that conditions vary and that varying conditions will justify different construction in different locations; that in places of light traffic a needlessly expensive construction would be prohibitory to street railway operation. That condition nobody, I think, would contemplate. In situations of heavier traffic heavier construction is justified, but what is the best construction is what we are all, no doubt, trying to arrive at.

The Reason for Railroads.—"To go back into history, I may remind you of what brought the railroad into existence, *viz.*, the lack of a surface suitable for a vehicle to carry sufficient people fast enough. For that reason the pioneers in railroading provided a surface for that purpose, out of which has grown the wonderful network of railroads. Following them, less than thirty years ago, the electric railway came into existence and has also performed its functions, and developed its wonderful system of railways in all centres of population and inter-connecting these centres and ramifying out from these cities through the adjoining country. If you stop to think, you will realize the immense mileage of the street railways, the figures for which I am not prepared to quote, the wonderful development and expansion which they have enabled the cities to enjoy and the tremendous increase in the value of properties and all that they have contributed.

"I have no doubt that you will all give due consideration to the interest of the street railways and will also have in mind that, with the advance in the cost of material, labor and everything else, the street railway is doing what it can to maintain its end and give the best possible service for popular fares.

"I noticed the other day a remark made by an engineer in New York that, if all of the railways in that city were to stop for some reason, the people would not only be unable to go to their work not only on account of the distance and time required, but also because there would not be room on the streets for the people who would want to use them. The essential value of railway transportation must be realized as well as that of vehicular travel.

"In the development of construction we have gone through a great many experimental phases, which we must not forget, and many times we have arrived at what we thought was perfection. Some years ago we laid a track on a girder or stringer of concrete. That was considered a wonderful development. I don't know whether it is still in use anywhere. It may be, but I know of so many places where it is disintegrated and failed that in the end it proved a great disappointment. Therefore, in arriving at the best and most permanent construction we must be very careful to be sure that we have the right thing before we cause the expenditure of vast sums of money to provide something which has not been established by time.

"I may assure you that the American Street Railway Association will lend its effort, interest and help in arriving at the solution of the problem and will be glad to collaborate with your association for the accomplishment of the desired end."

REPORT ON PREPARATION OF IRON AND STEEL SURFACES FOR PAINTING.

A COMMITTEE was appointed by the American Society for Testing Materials and the following report was sent in, as taken from advance copy published prior to the annual meeting of that society in June, 1916:—

The preparation of a steel or iron surface for painting should be such as will secure proper adhesion of paint to that surface. It is improbable that paint ever acts chemically on these metals; and the persistence of paint on iron is primarily a matter of adhesion, which may be lessened or destroyed by (1) any unsatisfactory surface, and (2) by the entrance or intrusion of solid or fluid material between the paint film and the metal.

Recently rolled steel or iron is covered with a mill scale of anhydrous oxide, and if painted at once, the paint never touches the metal, but is applied to the mill scale. If this mill scale ever comes off, the paint comes with it, sometimes in scales. If the metal begins to rust by access of air and moisture, the rust penetrates under the mill scale and loosens it. Ordinary rust is hydrated oxide, and stimulates further corrosion, but the anhydrous mill scale does not, and it is objectionable because it may crack off by unequal expansion or from other causes.

In addition to mill scale and rust, other objectionable surface coatings which are frequently encountered are dirt, grease, oil, water and frost.

In considering methods for preparing these surfaces for painting it is well to take account of the methods used to secure the adhesion of substances other than paint to iron or steel. Such cases are, for example, the electro-deposition of copper or other metals; the plating of iron by molten metal; the coating of steel or iron with a vitreous enamel, which is practiced in making enameled vessels for cooking and the like; and the application of varnish enamels, such as are used on bicycles and many other metal surfaces. In all these processes it is essential that the adhesion should be perfect; that is, that the coating should wear off from the outside, not peel off from the metal; and this is what is desired with paint. In all these cases, it is universally believed to be necessary that the coating material should come in actual contact on all parts of the surface with the actual metallic surface of the iron or steel; the latter must be freed from all dirt and grease, and from all scale and rust, before the coating is applied. This is done by (1) cleaning the surface with chemically active liquid, such as sulfuric acid, (2) by the sandblast, and (3) by other mechanical means, such as filing or polishing with an emery-belt, and the like. Unless this is done, it is found that the superimposed coating is likely to scale or flake off.

The thorough methods of cleaning by sand-blasting and pickling can be and sometimes are applied to structural and car steel for painting and for repainting, and undoubtedly are the best methods known for the purpose. They are, however, much more expensive than the ordinary method which consists in scraping, wire-brushing and wiping grease and oil spots with gasoline or benzine.

The sand-blasting method has the advantage over the pickling method in that it is more general of application, the pickling method being confined to the shop and generally to the material before assembling. It may, however, be of interest to know that good authorities maintain that iron or steel cleaned by pickling holds a coating more securely than that which is sand-blasted, and that this is owing to the rougher surface, viewed with a microscope, of the acid-etched metal.

The scraper and wire brush do not remove the firmly adhering mill scale, in consequence of which most of the structural and freight-car steel is painted over mill scale. It must be remembered that all platers and enamelers insist absolutely on the complete removal of mill scale; therefore it must not be regarded as harmless. It certainly is less dangerous than ordinary rust.

Builders of ships for service in sea waters have frequently required the pickling or sand-blasting of the steel parts which are to be submerged, in order to remove the mill scale, and it is the common practice to do likewise for steel passenger-car bodies. The removal of mill scale at the expense of incipient rusting is also sometimes attempted by the erection of steel structures without paint and allowing them to stand exposed to the weather for several months before painting.

In addition to cleanliness of surface, freedom from dampness, severe cold and frost is considered essential to the proper adhesion of paint. This may be accomplished by painting outdoors only in warm, dry weather, or by keeping the material under cover in warm, dry air during the process of cleaning and painting. Heating of surfaces is also resorted to.

While for some purposes, such as sea-going ships and passenger-car bodies, there seems to be little question as to the final economy of incurring the additional first cost of the more thorough methods of cleaning, the economy of such methods for ordinary steel structures and freight cars is not so certain.

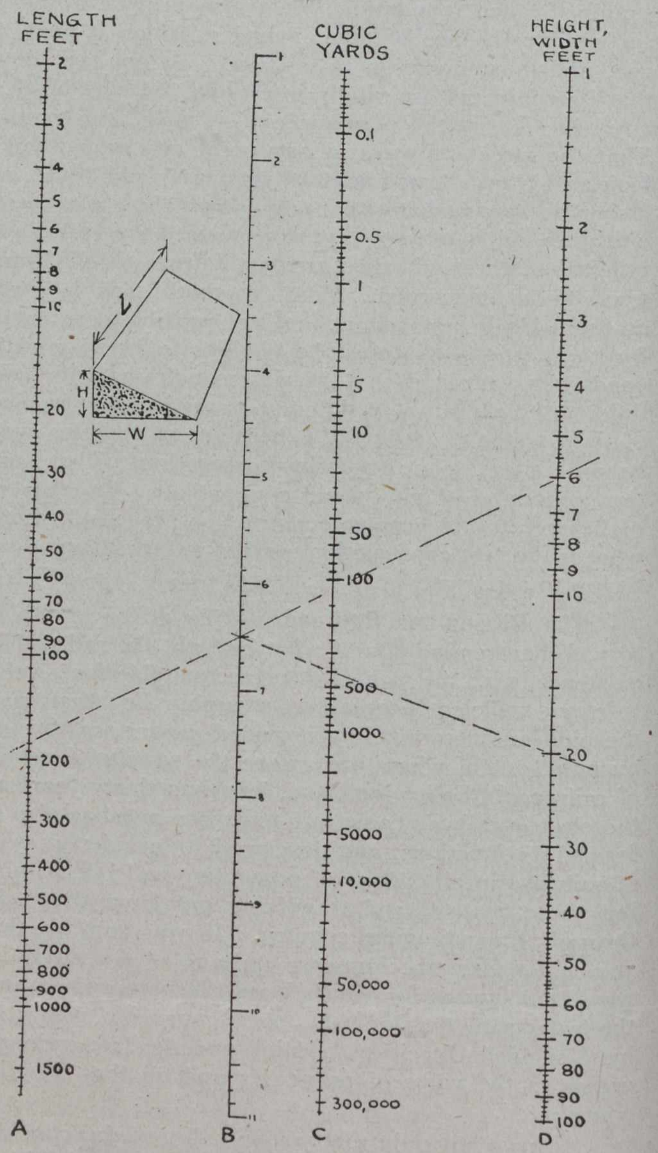
The sub-committee has considered the subject carefully at three meetings and recommends a series of panel tests to demonstrate, if possible, the relative merits of the different methods of preparing the surfaces and the effect of extremes of atmosphere and temperature conditions during painting. A programme for such a series of tests has been prepared.

Various recipes are given from time to time in architectural and building papers for the preparation of paint to be applied to cement and concrete surfaces. There is, however, nothing better than the zinc sulphate treatment. The surface is first washed with a solution of zinc sulphate in the proportions of 3 lb. to a gallon of water, and when dry can be painted with ordinary oil paint without danger of discoloration. The zinc sulphate changes the lime of the cement into calcium sulphate, while zinc oxide is deposited in the pores of the cement. As the two products of the chemical reaction are permanent and inert substances, familiar to plasterers and painters, they have no harmful effect upon either the surface or the paint subsequently applied.

HANDY EXCAVATION CHART.

THE accompanying chart will be found of value to those of our readers who are more particularly interested in engineering contracting work. The sketch drawn in the chart gives the idea of the section clearly.

For example: How many cubic yards in a triangular excavation where L = 180 feet (see column A); H = 6 feet (see column D) and W = 20 feet (see column D)?



Excavation Chart for Triangular Sections.

The dotted lines drawn across the chart show how it is done. First join the 180 and the 6 and find the intersection with column B. Then from that intersection point run over to the 20 and the intersection with column C gives the answer—400 cubic yards. It is unnecessary to do figuring; the chart does it all.

CANADIAN NORTHERN RAILWAY MILEAGE.

Among the interesting items included in the annual report of the Canadian Northern Railway Co., is the statement of mileage which totals 7,761 miles, distributed among the provinces and states as follows: Nova Scotia, 380.76; Quebec, 626.77; Ontario, 1,480.65; Manitoba, 1,983.46; Saskatchewan, 2,111.85; Alberta, 962.19; State of Minnesota, 215.42.

Editorial

EMPLOYMENT FOR RETURNED ENGINEERS.

Since the outbreak of war many readjustments have been made necessary. These readjustments have invaded the field of the civil engineer in a very peculiar sense. The stress of war and the necessity of providing for its conduct has undoubtedly meant that money which in normal times might be used for public works of all kinds has been diverted to the coffers of the war chest. At the same time the volume of undertakings in which the civil engineer is involved has been rather striking during the past two years.

While it would be foolish to prophesy as to how long the war will go on, it is well that those interests which are concerned should begin now to provide ways and means by which those of the profession who are now on the firing line can be assimilated immediately they are liberated after peace has been declared.

In view of these facts it is incumbent upon public authorities of all kinds, railway companies and large corporations generally who can employ engineers, to make all preliminary arrangements possible to take care of these men when they return to Canada.

Engineers in large numbers have responded to the call for volunteers. They have done very wonderful work at the front. Is it too much to ask that those at home give this matter some serious thought?

It may be urged that the various local financial situations do not permit any such anticipatory measures. Nevertheless, the government might be called upon to advance the necessary loans to municipal authorities and public corporations in an effort to carry out undertakings of a profit-yielding character rather than the organization of benevolent schemes for the help of men who will be unable to find an outlet for their ability immediately upon their return to Canada.

VALUE OF COST DATA AND ITS COMPILATION.

Most engineers will recognize the desirability of comparison in costs on similar work under different organizations, and feel themselves under distinct obligation to the engineer who, recognizing how acceptable such data are, is willing to turn them over to his fellow engineers either through a paper or by publication in the engineering journals.

To learn from one's own experience takes time and sometimes costs money, and even then the experience of an individual is limited.

The periodical publication of cost data is simply the doing of one's "bit" toward a common good. It is unthinkable to expect one engineer to give inside information on his own costs unless he has reason to believe that some other engineer who has been engaged in similar work and perhaps under somewhat similar circumstances is willing in his turn to give him the benefit of his experience.

In preparing cost data there must be a due regard for comprehensiveness, clearness, conciseness, low expense of keeping, ease of keeping, regularity of compilation and last, but not least, their periodical publication.

CONSERVATION.

Slowly but surely the people of Canada are coming to a more intelligent appreciation of what the development of our water powers and the establishment of a sane and efficient forestry policy means. The question of conservation is one which comes a great deal closer to the province of engineering than many have realized. Public opinion is, however, being safely directed and is giving itself expression in the enactment of laws which, if enforced, will go a long way toward correcting the altogether too common opinion that conservation means merely to "save" and nothing more. Nothing could be further from the truth. Such saving would be, in reality, the very embodiment of wastefulness.

It might almost be said that all our resources, no matter what their character, are valuable just to the extent that they are used for the public benefit. True conservation, therefore, is the use of our resources to the best advantage so as to render the greatest good. Conservation of a tree, for instance, means to use the tree at the time it is most valuable and to adopt at the same time the best and most economical method of reproduction.

STUDY OF ACCIDENTS.

The following facts taken from the report of the Bureau of Accidents Prevention of the Portland Cement Association will be of interest:—

At the end of 1915 the Bureau had on file over 11,000 reports of accidents, 135 of which covered fatalities which occurred during 1913, 1914 and 1915. From a summary comparison of the accident figures for the three years in question it was found that in 1913 they averaged 62 7/10 per cent. per million barrels of cement produced. In 1914 the average was 74 8/10 per cent. and in 1915 64 per cent. In 1915 one hundred and thirty-five fatalities were reported to the Bureau, which shows a reduction of 50 per cent. in fatal accidents when compared with the figures for 1914. The majority of the causes of the accidents, as shown by a table in the report, were of the usual character, such as caught between belt and pulley, run over by railroad car or box car, drowning, falls from various distant heights, etc. From the summary it will be found that the commonest causes of the accidents, according to their classification, are falling objects, flying material and breaking or slipping of machine parts or objects. There is a marked difference shown in 1915 under the heading "caught between parts." This is due largely to the fact that much progress on the mechanical safe-guarding of plants has been made.

A distribution of fatalities for 1915 shows that the greatest number of accidents or fatalities occurring on any single day of the week was eight for Monday and also on Friday. These days are also charged with the greatest percent. of the total time lost on account of accidents.

Another table shows the distribution of accidents according to the hours of the day in which they occurred. In 1913 ½-hour periods were employed, but it was decided that the ½-hour periods were too small, and on account of the fact that the majority of the reports came in with even hours mentioned on them there was no benefit derived from the use of half-hour periods. The table for 1915 was prepared on the one-hour basis. It was found that the greatest number of accidents occurred between 9 and 10 in the morning and between 2 and 3 in the afternoon.

A glance at the chart which accompanies the report shows that accidents are consistent in their occurrence as regards their distribution throughout the day. Another table in the report shows the length of time the injured employees had been employed on their particular operations before the occurrence of the accident. In 1915 workers employed four weeks or less sustained 792 accidents, which is in excess of any other class shown by the table. This clearly indicates the importance of the proper education of new employees before allowing them to take up their duties. The hiring of new men is costly at best and if new workers are not properly instructed in the performance of their duties, the probability of accident during the periods of their employment is greatly increased.

In 1915 the greatest number of accidents occurred to those who had been at work four hours. The 1914 figures show more accidents to have occurred to employees having been at work two and three hours, while 1913 figures give three and four hours as having the high value. The report, furthermore, shows a decrease in accidents occurring to employees under 22 years of age and an increase in employees between the ages of 30 and 40.

LETTER TO THE EDITOR.

"The Technically Trained Man in Business."

Sir,—I have read with interest your recent editorial on "The Technically Trained Man in Business," (August 3rd issue *The Canadian Engineer*). I am myself a young engineer quite recently graduated from a well-known Canadian university, and I realize quite fully that I know nothing of business methods and very little of business law. I am not at all favorably impressed with the means adopted in some, at least, of our universities, to make the student familiar with business methods or business law. Perhaps, being but a mere student, I am not at all qualified to criticize the methods of my Alma Mater, but it appears to me that a course in business law which compels a student to take dictation in that subject, for one hour per week, for one year, at a furious rate of speed, and subject to all the errors incident thereto, and without any opportunity whatever for discussion, is, at any rate, not all that it should be. As far, however, as we young graduates are concerned, this is past and gone. What concerns us, as young engineers, is how we can acquire this information. Unfortunately, some of us are not, on account of our technical duties, thrown into contact with the business world, and we are therefore left to grope for ourselves. It seems difficult to make much progress.

Personally, beyond reading somewhat carefully such Acts of Parliament as the Bank Act, the Joint Stock Companies' Act, etc., I have accomplished nothing. I would appreciate, and I believe that many young graduates would also appreciate, a series of articles in your excellent magazine dealing with these subjects—business methods and business law. It would doubtless be impossible to go very fully into these subjects, but at any rate many useful suggestions as to how and where to acquire information along these lines could be given; and possibly some of the more obscure problems could be dealt with more or less at length.

R. C. McCULLY.

Bulyea, Sask., August 14, 1916.

PERSONAL.

W. E. SEGSWORTH, Toronto, has been elected a member of the Institution of Mining and Metallurgy.

JOHN H. GRAY, A. W. McVITTIE and MARTIN H. RAMSAY, all of Victoria, B.C., have been authorized to practice as land surveyors in British Columbia.

J. A. JOHNSTON, who for some time has filled the position of district inspector of the Hydro-Electric Commission at Brockville, Ont., has been appointed manager of the Light and Power Department of that town.

Major W. N. ASHPLANT, former city engineer of London, Ont., who was recently slightly wounded in the ear by a shrapnel fragment, and who returned to duty within a few days, has been officially reported as wounded and missing.

K. B. KUNDSEN, B.Sc., of London, England, has joined the Toronto staff of L. B. Mouchel & Partners, engineers. Mr. Kundsen is a member of the Danish Society of Civil Engineers. He has lived in England for the past three years, and recently was captured by the Germans while en route to Denmark for a vacation. As he was a citizen of a neutral country, however, he was soon released.

OBITUARY.

Major W. A. CASEY, a well-known civil engineer of Victoria, B.C., has been killed in action.

FRED. WALKER, first superintendent of the Lethbridge division of the C.P.R., died recently at Tacoma, Wash., after a long illness.

H. L. WILLIAMS, of Leamington, Ont., who for many years was engaged in the building trade, recently passed away at the age of 84.

A. J. L. EVANS and GEORGE REVELL, who went to the front with the mining engineers from Kootenay, B.C., have both been killed in action.

JOSEPH NEWMAN, resident of St. Catharines, Ont., for the greater part of his lifetime of sixty years, and a former member of the contracting firm of Newman Bros., died on September 21st after a lingering illness.

J. H. LEAHY, contractor, died recently at the Royal Victoria Hospital, Montreal. Mr. Leahy had been associated with works in that city for many years. He had the contract for the paving of Notre Dame Street through Maisonneuve and for the construction of many city drains and sewers.

JOHN BANNERMAN, a well-known resident of Ottawa, passed away recently, following an operation. He was born in Scotland 66 years ago and came to Canada at an early age, settling in Ottawa, where he resided ever since. Formerly he was a member of the firm of Powers and Bannerman, and was the constructor of the first pipe for the Ottawa waterworks system.