PAGES

MISSING

SUBSCRIPTION RATE: $3 a year; $1.75, six months; $1, three months; postpaid anywhere.

The Canadian Engineer

Published Weekly. Established 1893.

Issued in the Interests of Canadian Civil Engineers and Contractors

This field includes chiefly the men who are engaged in municipal, railroad, hydraulic, structural, highway and consulting engineering; surveying; mine management; contracting; and water works superintendence.

Index to Volume 32

January 1 to June 30, 1917

Published every Thursday by THE MONETARY TIMES PRINTING CO. OF CANADA, LIMITED
HEAD OFFICE: CORNER CHURCH AND COURT STREETS, TORONTO, ONTARIO
JAMES J. SALMOND, President and General Manager. ALBERT E. JENNINGS, Assistant General Manager.
Telephone Main 7404; Branch Exchange connecting all Departments.
Cable Address: "Engineer, Toronto."
WESTERN CANADA OFFICE: 1208 McArthur Building, Winnipeg. G. W. GOODALL, Western Manager.

The Canadian Engineer

INDEX TO VOLUME 32

January 1 to June 30, 1917

A

B

C

*Illustrated. **Letter to the Editor. †Editorial. ‡Author of Book Review. §Obituary

D

E

F

G

H

I

J

*Illustrated. **Letter to the Editor. †Editorial. ‡Author of Book Review. §Obituary.

*Illustrated. **Letter to the Editor. †Editorial. ‡Author of Book Review. §Obituary.

S

T

U

V

W

*Illustrated. **Letter to the Editor. †Editorial. ‡Author of Book Review. §Obituary.

Y

Z

*Illustrated. **Letter to the Editor. †Editorial. ‡Author of Book Review. §Obituary.

The Canadian Engineer

A weekly paper for Canadian civil engineers and contractors

UNIVERSITY BRIDGE, SASKATOON

TEN-SPAN CONCRETE BRIDGE IS 1,407 FEET BETWEEN EXTREME ENDS OF APPROACH WALLS, AND IS BUILT TO GRADE OF 2.88 PER CENT.—SUMMARY OF COSTS, SPECIFICATIONS AND CONSTRUCTION METHODS.

By C. J. YORATH, A.M.Can.Soc.C.E., A.M.I.C.E.,

City Commissioner, Saskatoon, Sask.

IN the early part of 1913 an agreement was entered into between the city of Saskatoon and the provincial government of Saskatchewan, whereby it was agreed that the latter would construct a reinforced concrete bridge over the Saskatchewan River, running south-easterly from 25th Street and Spadina Crescent to the corner of College Street and Clarence Avenue. This site was chosen principally with the idea of providing more direct communication between the University of Saskatchewan, which is located on the east bank of the river, and the central business portion of the city on the west bank. The only other means of access across the river is by means of a steel truss bridge, which, on account of its width (25 ft. 6 ins.) often becomes congested with traffic.

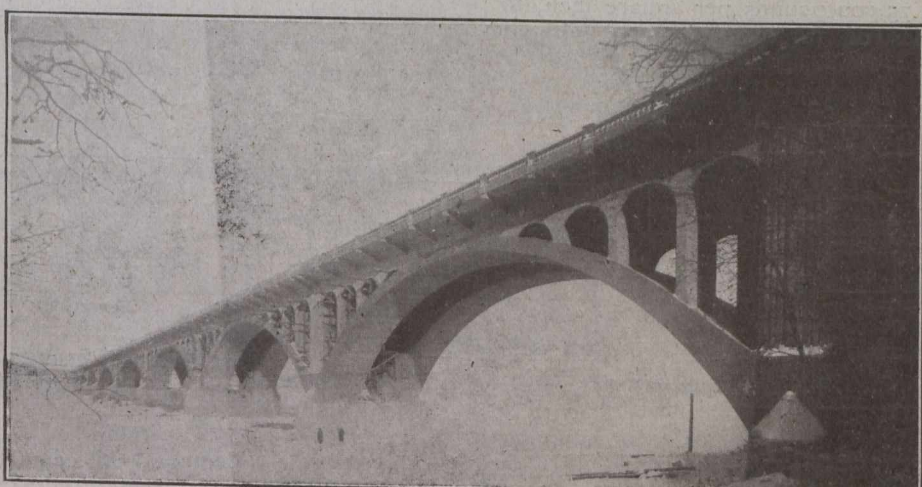


Photo February 1st 1916

View of Bridge from East Bank of River, Showing Cantilever Sidewalk.

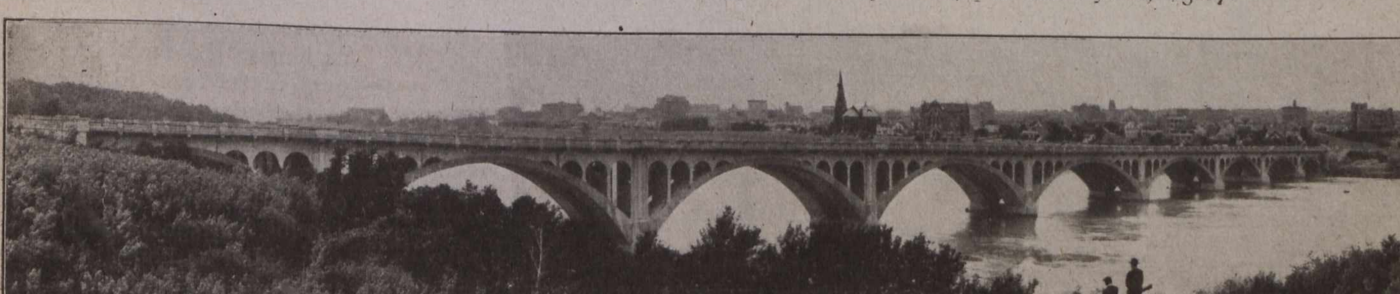
The agreement between the city and the government provided for a bridge 62 feet wide, with two sidewalks 8 ft. 6 ins. wide, and a carriageway 45 ft. wide, to be designed to carry a double line of street railway tracks, and provided with a conduit to accommodate sewer, water and electric mains. It was originally intended to carry the piers of the bridge upon reinforced concrete piles, but it was found upon trial-holes being made, that immediately below a strata of gravel and boulders averaging in depth about five feet, and which formed the bed of the river, there was a strata of unknown depth of hard, blue clay which would make an excellent foundation for the load to be carried by each of the piers.

A contract was let to the R. J. Lecky Company in June of 1913 which amounted to $240,004.86 at the unit prices quoted in the tender, the provincial government undertaking to supply the cement and all steel work, which was estimated to cost $158,000, so that the total estimated cost was $398,004.86. The highest tender received, including the cost of material to be supplied by the government, amounted to $755,594.

Unit Prices.—The unit prices quoted in the accepted tender were as follows:—

For concrete in piers and abutments below the springing line of arches and in retaining walls, placed, waterproofed and finished as specified, per cubic yard, $4.77.

For all concrete above the springing line of arches (except concrete in fence and posts or concrete used in pavement or otherwise paid for) furnished waterproof and placed as specified, per cubic yard, $5.87.



View from University Campus, October, 1916. *Photo by Finley, Saskatoon*

For concrete fence above line of sidewalk, as shown on plan, per lineal foot, $1.

For rip-rap, if required, price per cubic yard in place, $18.

For wooden piling under foundations and required in finished structure, price per driven foot, $.40.

For concrete piling under foundations and required in finished structure, price per driven foot, $1.05.

For dry excavations measured as specified to get proper foundations, per cubic yard, $.90.

For wet excavations, including cofferdams measured as specified to get proper foundations, per cubic yard, $7.07.

For sidewalk on approaches and over earth-filled parts of bridge, per square foot, $.15.

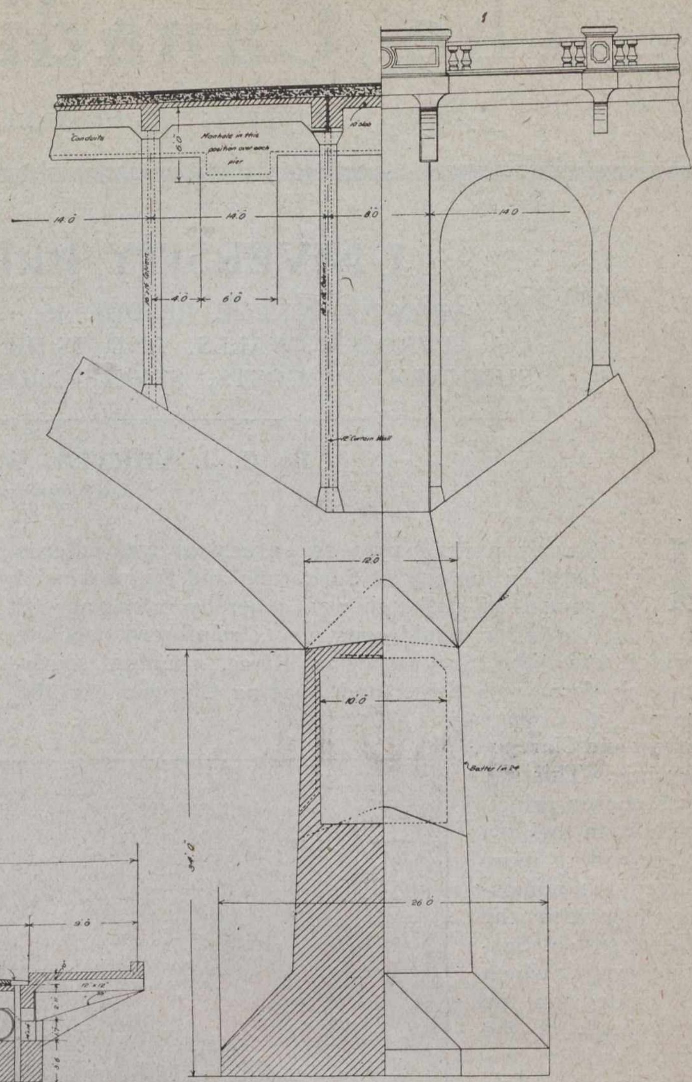
For curb and gutter in approaches and over earth-filled parts of bridge, $.35.

For embankment in approaches, $.35.

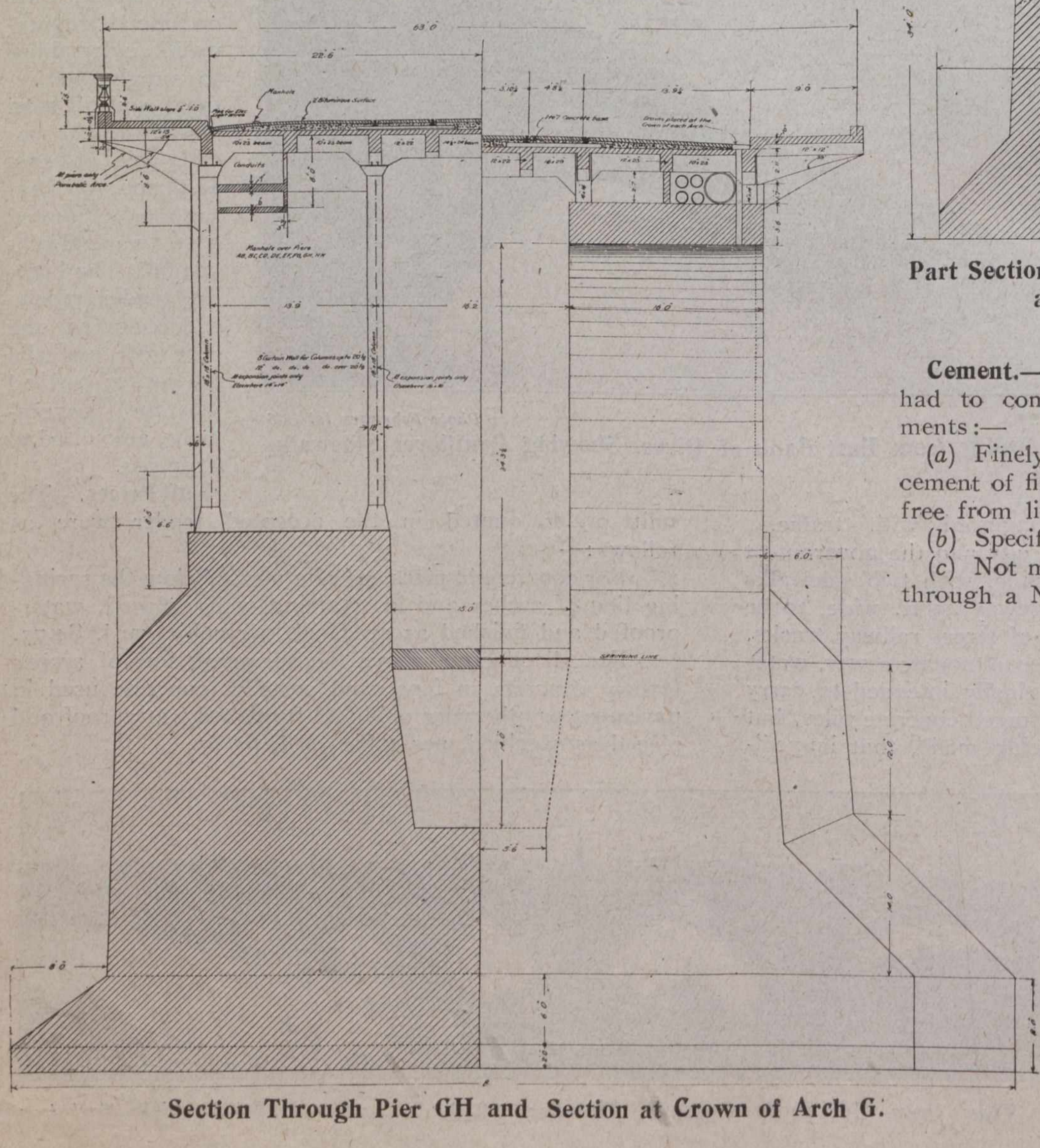
For the supply of all material not otherwise provided for; falsework and centering; conduits, manholes, manhole covers, ladders and drainage system complete; and for all labor and material required to carry out the work not otherwise paid for,—the lump sum of $79,141.

The specification provided for the following:—

Steel.—To be soft or medium having an ultimate strength of 55,000 to 75,000 pounds per square inch, to bend cold 180 degrees on its own diameter without cracking.



Part Section Through Centre of Pier GH and Part Elevation.



Section Through Pier GH and Section at Crown of Arch G.

Cement.—All cement used in the works had to comply with the following requirements:—

(*a*) Finely ground, fresh-burned Portland cement of first quality and approved brand, free from lime and other impurities.

(*b*) Specific gravity not less than 3.10.

(*c*) Not more than 1% to be left unsifted through a No. 50 sieve; not more than 8% on a No. 100 sieve; and not more than 25% on a No. 200 sieve.

(*d*) Tensile strength to be not less than 150 lbs. per square inch and not more than 450 pounds after a period of 24 hours. Briquettes made of one of cement to three of sand to develop the following strength:—

1. After 24 hours in air and 6 days in water, not less than 150 lbs. per square inch.
2. After 24 hours in air and 27 days in water, not less than 200 lbs. to the square inch.

Neat cement to develop its initial set in not less than 30 minutes and its final set in not less than three hours.

Loading for Floors.—(*a*) On each car track a concentrated load of 100 tons carried on two trucks having a wheel spacing of 5-15-5 feet between centres of wheels along rails, each wheel carrying 12.5 tons.

Showing Timber Centering for Arches Carried Upon Piles, Trestle Carrying Concrete Delivery Pipe, and Reinforcing for Spandrel Columns.

(*b*) On remaining part of floor surface a concentrated load of a 25-ton traction engine, wheels 8-foot centres and 10 feet between axles, two-thirds of weight on rear wheels which are 30 inches wide.

Temperature Variation.—A range of temperature from — 50 degrees to + 90 degrees Fahrenheit was assumed for the calculations of temperature stresses.

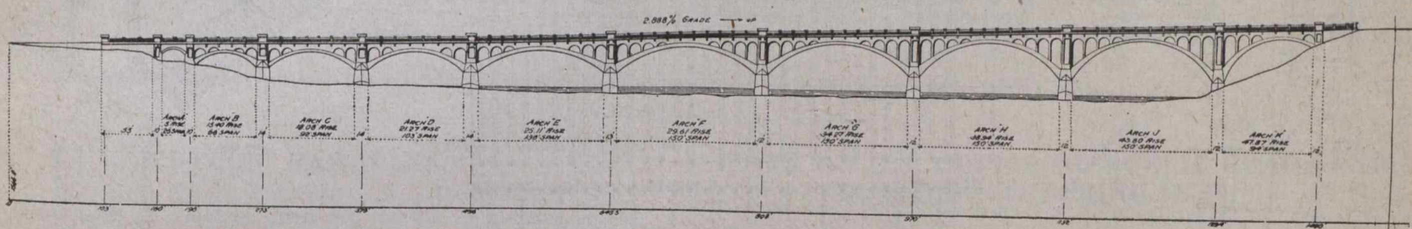
Quantities and Dimensions.—The total length of the bridge between the extreme ends of wing approach walls is 1,407 feet and is built to a grade of 2.88 per cent., the east end of the bridge being 35 feet higher than the west end.

There are ten arches, with the following spans:—

Clear span.	Centre to centre of piers.
25 ft. 0 in.	34 ft. 0 in.
66 ft. 0 in.	78 ft. 5 ins.
92 ft. 0 in.	106 ft. 0 in.
103 ft. 0 in.	117 ft. 0 in.
136 ft. 0 in.	149 ft. 0 in.
150 ft. 0 in.	162 ft. 0 in.
150 ft. 0 in.	162 ft. 0 in.
150 ft. 0 in.	162 ft. 0 in.
150 ft. 0 in.	162 ft. 0 in.
94 ft. 0 in.	106 ft. 0 in.
Total span	1,238 ft. 5 ins.

The bridge contains 23,000 cubic yards of concrete and 1,015 tons of steel.

The larger spans contain in the arches sixty-four 1¼-inch diameter rods having an area of 78.54 square inches, one-half being extrados and the other half intrados rods.



Elevation, University Bridge, Saskatoon.

(*c*) On sidewalks a uniform load of 150 lbs. per square foot.

Loading for Arch Rings.—(*a*) A uniform load of 200 pounds per square foot for the centre 17 feet of the bridge.

(*b*) A uniform load of 150 pounds per square foot for the remaining floor surface of the bridge.

(*c*) A uniform load of 150 pounds per square foot for sidewalks.

Working Stresses.—The allowable stresses in the materials not to exceed the following:—

Compression in concrete, 600 pounds per square inch.

Tension in concrete, not reinforced, 0 pounds per square inch.

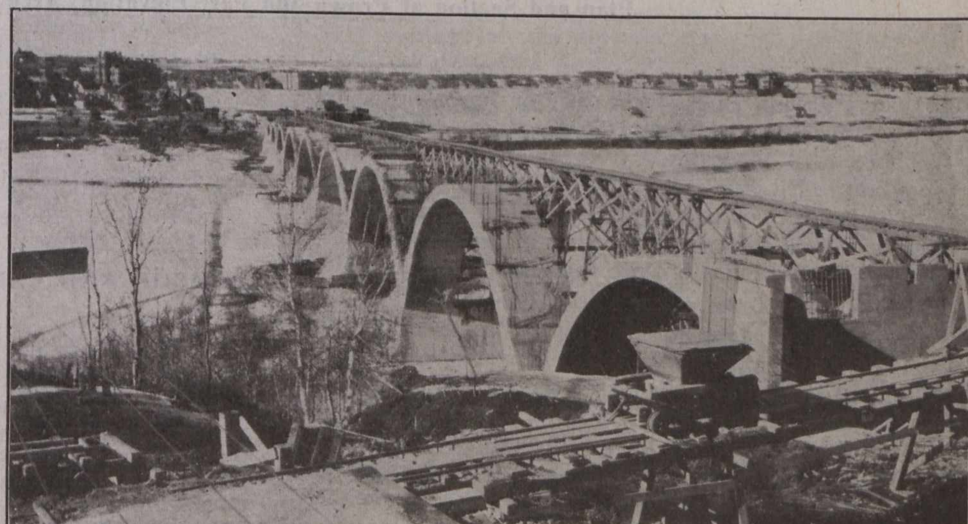
Shear in concrete, 50 pounds per square inch.

Tension in steel, 13,000 pounds per square inch.

Compression on steel, not more than 15 times that of concrete.

Foundation Pressures.—Four tons to the square foot.

The area of steel in the beams varied from 8⅝-inch diameter rods having an area of 2.45 square inches in 12-in. x 23-inch beams to 10⅞-inch diameter rods having an area of 6.01 square inches in 16-in. x 29-in. beams.



East Abutment. Centering Removed. Before Pouring Spandrel Columns or Floor System.

The area of steel in the floor slab averaged .47 square inches of steel per lineal foot; also temperature rods ½-inch diameter, spaced 10 inches centre to centre.

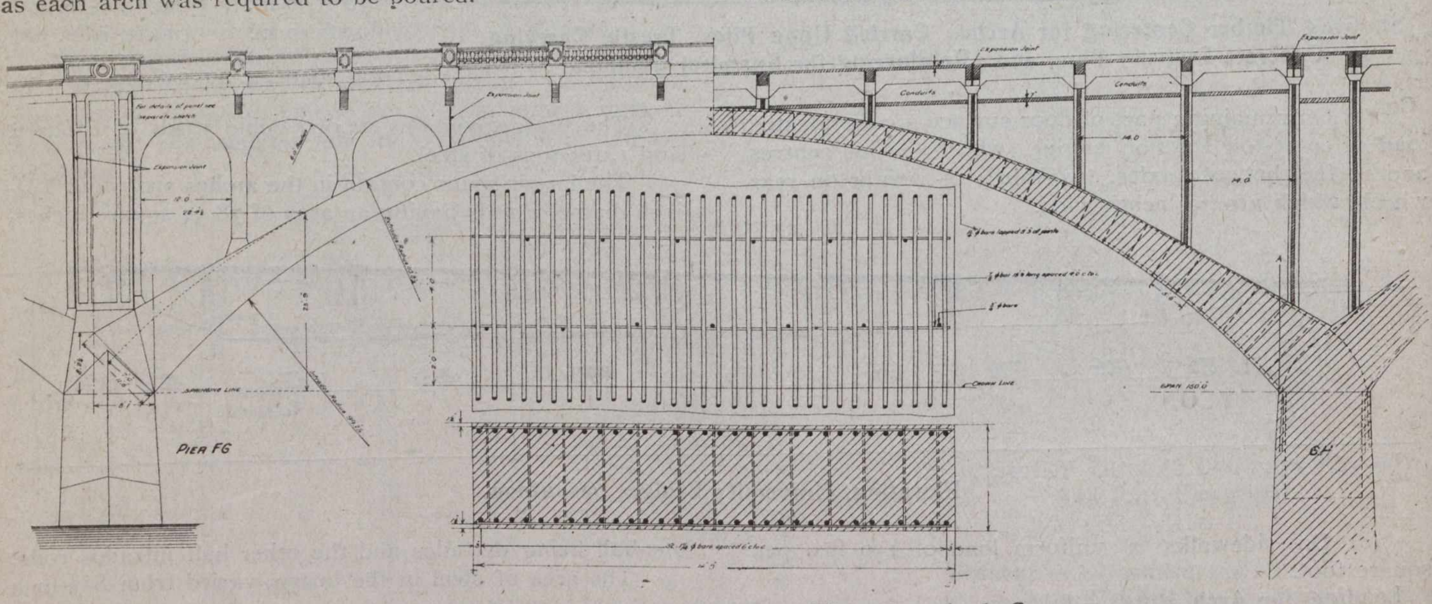
The extreme dimensions of foundation for largest piers was 84 ft. x 26 ins. and the average dimensions of main body of pier (without ice breaks) 48 ft. x 13 ft., and the height from base to springing line of arches, 34 feet.

Construction Methods.—The concrete for piers was poured when the river was frozen over. Inside coffer-dams were constructed with Wakefield sheet piling made on the job and consisting of three layers of 2-inch planks spiked together so as to make tongues and grooves. Generally, two rows were driven about two feet apart and were puddled between with clay.

The concrete in arches was poured from a pneumatic mixer fixed on the east bank of the bridge. The 94-foot arch was poured first, and then a discharge pipe from the mixer was conveyed upon a timber trestle and extended as each arch was required to be poured.

layers of tarred felt. The beams, which run transversely to the bridge, are double at expansion joints. One beam was poured monolithic with the columns supporting it, and the other beam is separated from the column by a bronze plate set in that half of the column under this beam. The two beams are separated by several thicknesses of tarred felt. The cantilever arm opposite this joint is made monolithic with one of these beams, and separated from the other by a paper-filled joint. The fence and outside longitudinal stringers also have an expansion joint at the end of the cantilever arm, the lower bronze plate fixed to the top of the cantilever arm, and the other plate attached to the under side of one of the outside stringers at the same point.

The ends of stringers have an expansion joint somewhat similar to a tongue and groove, with layers of tarred felt between. Over the beams and cantilevers the expansion joint is carried up through the floor slab and sidewalk with several layers of tarred felt. Through the paving base the expansion joint is formed by a joint filled with sand and bitumen.



Plan and Section at Crown and Part Elevation, Arch G.

The mixer used was a vertical cylinder with cone-shaped ends, having an air-tight door on top, worked by air valves through which ingredients were introduced. At the bottom of the cylinder there were several 2-inch air pipes controlled by a valve. The air for the mixer was supplied by two air compressors at a pressure of 90 pounds per square inch at the commencement of forcing the concrete from the container through the delivering tube.

The operator of the mixer stood beneath the loading platform, and after allowing the charge to be emptied into the container, along with sufficient water, closed the top door and then turned in air at the 90 lbs. pressure. After a few seconds he opened the tower door and allowed the charge of concrete to be blown through the delivering pipe to the receiving hopper. The concrete for spandrel columns was poured from wheelbarrows loaded from the receiving hopper fed by the pneumatic mixer.

Owing to the great variation in temperature in Saskatoon, ample provision had to be made for expansion joints. The upper portion of the superstructure, including the floor system, is divided transversely by expansion joints at each side of the pier and at two other places in each span. At these points the spandrel walls are divided by a vertical joint, V-shaped, which is filled with several

The total cost of the bridge was approximately $520,000, two-thirds of which was borne by the provincial government and the remaining third by the city. The structure was designed in the Highways Department of the Provincial Board of Highway Commissioners.

HARBOR AND CANAL EXTENSION IN SWEDEN.

The construction of the new Trollhätta Canal has led to a series of other work in connection with harbors and waterways. It is now being urged that the canal between Lake Vänern and Lake Vättern should be improved so as to have the same depth and breadth as the new Vänern-Gothenburg waterway. At Kristinehamn extensive harbor works have been going on for several years, entailing an expenditure of some 4,000,000 kroner. The work is now completed and the new harbor is about to be opened. The depth is 4 m. (13 ft.), and quays, store accommodation and ample railway connections have been provided.

Since the fire of 1906 the city of San Francisco has issued a total of 66,278 permits for building construction, the value of which was $292,846,885. The figures include eighty-three exposition buildings.

CANADA'S ELECTRIC POWER EXPORTERS.

The growing and somewhat insistent demand for power on the part of the municipalities of Ontario for industrial and domestic purposes has brought the question of a reduction of the export quantities granted by license to the hydro-electric companies at Niagara Falls before the department of inland revenue for revision, states Mr. J. V. Vincent, in his annual return. The electricity exportation act provides that licenses to export "shall be revocable upon such notice to the licensee as the governor-in-council deems reasonable in each case." Under the circumstances a yearly reduction in the quantities to be exported might be deemed a reasonable method of putting an end to the export.

Electrical standardizing laboratories have been established at Ottawa and Vancouver, where the substandards used by the department's inspectors are adjusted and standardized periodically, also such electrical instruments as may be presented by the general public. The equipment for another similar laboratory at Winnipeg has been provided and awaits suitable accommodation for the installation thereof.

The revenue collected from weights and measures inspection was $112,136; gas and meter inspection, $46,034; electric meter inspection, $70,051; making a total from this source of $228,221.

The amount of electrical energy produced for export and consumption in Canada for the year ended March 31st, 1916, is as follows:—

Name of contractor and place of business.	Units produced for export. K.W. Hours.	H.P. Years.	Units produced for use in Canada. K.W. Hours.	H.P. Years.	Total output of generating station or other source. K.W. Hours.	H.P. Years.
Canadian Niagara Power Co. .	400,521,090	61,289.01	11,178,910	1,710.59	411,700,000	62,999.60
Electrical Development Co. ...	34,652,000	5,302.56	395,346,500	60,497.02	429,998,500	65,799.58
Ontario Power Co.	199,135,160	30,472.28	616,834,440	94,389.93	815,969,600	124,862.21
*International Railway Co. ...	....	......	....	....	....	....
Ontario & Minnesota Power Co.	13,144,070	2,011.33	11,789,534	1,804.08	24,933,604	3,815.41
Cedars Rapids Power Co.	358,753,000	54,897.51	56,031,000	8,574.03	414,784,000	63,471.54
Sherbrooke Railway and Power Co.	230,820	35.33	8,605,200	1,316.79	8,836,020	1,352.12
Maine and New Brunswick Power Co.	3,075,893	470.69	242,437	37.09	3,318,330	507.78
Western Canada Power Co. ...	11,937,700	1,826.73	60,468,020	9,253.00	72,405,720	11,079.73
British Columbia Electric Railway Co.	330,626	50.59	68,470,689	10,477.60	68,801,315	10,528.19
Totals	1,021,780,359	156,356.03	1,228,966,730	188,060.13	2,250,747,089	344,416.16

*This company's load is used for electric railway supply, chiefly on Canadian side of boundary.

CEYLON RAILWAYS.

The length of railway open for traffic in Ceylon at the close of September, 1915, was 692½ miles, as against 672 miles at the corresponding year of 1914. The increase was due to the opening of a section of a line to Chilaw. In the course of 1914-15 rolling stock was increased by 18 new passenger and 186 new goods vehicles, including a number of tank wagons. The total expenditure to the close of September, 1915, including additional accommodation and improvements, amounted to £7,858,452. The Chilaw line was approaching completion in 1915, and it has been opened to Kochchikade for all descriptions of traffic. Steady progress was made with the Pelmadulla and Badulla extensions, especially on sections between Ratnapura and Dela and between Bandarawela and Ella. Work is in progress on Colombo station extension, and a large broad-gauge goods shed and connecting sidings have been brought into use. Earthwork is in hand on a main line duplication between Ragama and Veyangoda. A survey from Chilaw to Puttalam has been commenced, and a survey connecting the harbor line with the railway, via Mutwal, has been completed.

WORK OF U.S. OFFICE OF PUBLIC ROADS AND RURAL ENGINEERING IN 1915.

Over 4,942,000 sq. yds. of roadway, the equivalent of 561.9 miles of road 15 ft. wide, were constructed under the supervision of federal road engineers during the last fiscal year, according to the annual report of the Office of Public Roads and Rural Engineering, of the United States Department of Agriculture, just issued. This is more than double the mileage so constructed in previous years.

The yardage of the different types of surfacings constructed last year were as follows in square yards: Gravel, 1,848,280; macadam, 1,565,163; earth, 767,437; bituminous macadam, 310,643; sand-clay, 295,337; concrete, 84,454; bituminous concrete, 39,813; surface treatment, 30,835.

The roads constructed under supervision of the office include experimental roads, post roads, county roads, and roads in National Parks and forests. The expenditures of the office for this purpose were chiefly for engineering services and supervision, the communities, except in the case of forest and park roads, meeting the bulk of the costs for material and construction.

Ferro-vanadium exports from the United States have reached a total of 1,062,932 lbs. for the first eight months of this year and are constantly expanding. While the exports reached over 1,000,000 lbs. for the 12 months ended June 30th, 1916, they will amount to 1,594,396 lbs. for 1916, if maintained at the present rate.

The British Consul-General at Havre reports that the iron mines and blast furnaces of Caen (Normandy) are at present being worked by the Government, and that the industry, which was languishing for many years past, is now most flourishing. A large new plant of the most modern type is being installed, and the railway authorities are doing their utmost to assist the development of the industry. The output of iron ore in this district has now reached over 1,000,000 metric tons per annum.

Tungsten deposits in California, which remained practically unknown until the spring of 1916, have recently yielded considerable quantities of the mineral under the stimulus of the increased demand. The ore consists of scheelite associated mainly with garnet, epidote and quartz, and averages about 2 per cent. of tungsten trioxide

It appears that as a result of war conditions in Germany, carbon electrodes for furnaces are now being standardized. According to the new rules, round carbons for steel furnaces are to be made in sizes varying in thickness from 100 mm. upwards by steps of 25 mm., the tolerance varying from 3 mm. to 6 mm., according to size. For rectangular electrodes used with other kinds of furnaces, a standard size of 500 mm. square is being proposed. For the time being, the standard length is fixed at 2 m.

RECOMMENDED SPECIFICATIONS FOR REINFORCED CONCRETE DESIGN.

THE Portland Cement Association has issued recently specifications for reinforced concrete design. The specifications are based on the regulations in the building code recommended by the National Board of Fire Underwriters and the 1916 report of the Joint Committee on concrete and reinforced concrete. The specifications follow:—

Loads.—The schedule of superimposed uniform (live) loads per square foot is as follows:—

Roof .... pounds per square foot
.... floor .... pounds per square foot
.... floor .... pounds per square foot
Etc.

The roof slab, together with the roof beams and girders, shall be designed to carry the full live and dead loads. All floor slabs and beams shall be designed to carry the full live and dead loads. All girders under floors where the assumed live load is under 120 lbs. per square foot shall be designed to carry all the dead load and 85 per cent. of the assumed live load. No deduction for floor loads greater than 120 lbs. per square foot.

Every column, post or other vertical support shall be of sufficient strength to bear safely the combined live and dead loads transmitted to it.

The following reductions are permissible when the live loads are under 120 lbs. per square foot:—

For columns supporting roof: No reduction.

For columns supporting top floor: A reduction of 10 per cent. of the live load of the top floor may be made.

For columns supporting each succeeding floor: A reduction of 5 per cent. of the live load per floor may be made, but the maximum deduction for any floor shall not exceed 50 per cent. The reductions shall cease when a floor carrying a live load of more than 120 lbs. per square foot is reached.

For floors, beams, girders and vertical supports carrying machinery: At least 25 per cent. shall be added to the stresses from live loads to provide for effect of impact and vibration.

For sidewalks between the curb and building lines: Live loads shall be taken at 300 lbs. per square foot or a concentrated load of 5 tons at any point.

Materials.—Cement: The cement shall meet the requirements of the current Standard Specifications for Portland Cement of the American Society for Testing Materials.

Steel: The steel shall meet the requirements of the current Standard Specifications for Reinforcing Steel of the American Society for Testing Materials.

Aggregates and Water: The aggregates and water shall meet the requirements of the current specifications for aggregates and water of the Portland Cement Association.

Broken stone or pebble concrete not reinforced shall be assumed as weighing 144 lbs. per square foot, and when reinforced shall be assumed as weighing 150 lbs. per cubic foot.

Stresses.—Extreme fibre stress in compression for 1:2:4 concrete, 650 lbs. per square inch. For 1:1½:3 concrete, 810 lbs. per square inch.

In direct compression, 1:2:4 concrete, 450 lbs. per square inch. 1:1½:3 concrete, 560 lbs. per square inch.

Shearing stress in concrete when diagonal tension is not resisted by steel, 40 lbs. per square inch for 1:2:4 concrete and 50 lbs. for 1:1½:3 concrete.

Unit shearing stress when web reinforcement is used, 120 lbs. per square inch, of which 40 lbs. per square inch may be resisted by the concrete.

In proportioning the section of concrete for shearing stress, the effective depth from centre of compression area to centre of steel area shall be used. In beams of T-section the width of the stem shall be used. In ribbed floors the width of the stem and thickness of flange near supports shall be proportioned for cumulative shear on the span.

Bond stress between concrete and reinforcement, 80 lbs. per square inch for plain and deformed bars, based on actual section.

Tensile stress in steel, 16,000 lbs. per square inch, for main reinforcement. Tensile stress in stirrup steel, 10,000 lbs. per square inch.

Ratio of deformation of steel to 1:2:4 concrete shall be taken as 1:15 and to 1:1½:3 concrete shall be taken as 1:12.

Compressive stress in steel shall be equal to the compressive stress in the concrete multiplied by the ratio of deformation.

In continuous beams and slabs the extreme fibre stress in concrete in compression may be increased 15 per cent. adjacent to supports.

Bearing on a concrete surface having a total area of at least twice the area of the loaded portion may be taken at 700 lbs. per square inch for 1:2:4 concrete and 875 lbs. per square inch for 1:1½:3 concrete, and, generally, at 35 per cent. of the ultimate strength of the concrete used, when all other stresses are properly provided for.

Slabs, Beams and Girders.—The straight-line method shall be used in designing reinforced concrete.

The span length for beams simply supported, and for all slabs, shall be taken as the distance from centre to centre of supports, but need not be taken to exceed the clear span plus the over-all depth of beam or slab. For continuous or restrained beams built monolithically into supports, the span length may be taken as the clear distance between faces of supports. Brackets shall not be considered as reducing the clear span in the sense here intended, and the clear span between T-beams is the distance face to face of stems. Maximum negative moments are to be considered as existing at the end of the span as here defined.

The bending moments of girders, beams and slabs due to uniformly distributed loads shall be taken as not less than:—

1/8 WL, at centre when simply supported.
1/9 WL, over central support; and 1/10 WL, near the middle of the span when continuous for two spans only.
1/10 WL, over first continuous support; and 1/10 WL, near middle of end spans when supported at one end and continuous at the other end for more than two spans.
1/12 WL, at centre and intermediate supports when continuous over more than two supports.

The simply supported ends of spans shall be reinforced for a negative moment of not less than 1/16 WL.

W = total distributed dead and live loads.
L = length of span.

Floor slabs having supports extending along the four sides shall be designed and reinforced as continuous over the supports. For uniformly distributed dead and live loads the per cent. of load carried on the transverse span shall be found by subtracting 0.5 from the quotient obtained by dividing the length by the breadth. The remainder of the load shall be carried on the longitudinal span. If the length of the slab exceeds 1.5 times its width the entire load shall be carried on the transverse span. Using the values thus obtained, each set of reinforcement is to be calculated in the same manner as slabs having supports on two supports only, using correct depths to centre of steel. The amount of reinforcement thus determined per foot of width applies only to the middle half of the slab and one-half of this amount per foot of width may be used in each end quarter of the slab. The probable distribution of the loads from two-way reinforced slabs to supporting beams or girders shall be ascertained, and the moments in the beams or girders calculated accordingly.

The bending moments for slabs or beams with spans of unusual length or due to other than uniformly distributed loads shall be more exactly computed according to accepted theory.

In continuous slabs, beams or girders, full provision shall be made for the negative bending moments over the supports by placing sufficient negative reinforcement near the top of the members to resist the stress. This reinforcement shall pass beyond the point of contra-flexure in beams or girders and be anchored in the compression concrete of the member a sufficient distance to develop the full strength of the steel through bond stress. The critical section of continuous construction is over the support.

Where adequate bond is provided at junction between slab and beam, and the two are cast at the same time as a unit, the slab may be considered as an integral part of the beam, provided its effective width shall not exceed one-fourth of the span length of the beam, nor be greater than six times the thickness of the slab on either side of the stem.

In beams with T-sections the width of the stem only shall be used in calculating longitudinal shear and diagonal tension. An effective bond shall be provided at the junction of the beam and slab when the principal slab reinforcement is parallel to the beam, by the use of transverse reinforcement extending over the beam and well into the slab.

In the design of T-beams acting as continuous beams, sufficient compression area shall be provided on the under side at the support, either by the use of properly designed brackets or by embedding additional compression steel in the concrete extending to the point of inflection.

The minimum thickness of concrete floor slabs shall be 4 ins., and of roof slabs 3½ ins.

Cement or concrete floor finish shall not be considered in calculating the strength of floor members unless it be laid at the same time they are cast.

The design of composite floors consisting of rows of concrete tile or hard-burned terra cotta tile, gypsum blocks, sheet steel, or other approved fire-resisting material, separated by ribs, or beams of reinforced stone concrete, shall conform to all the provisions of these specifications so far as they are applicable. Not less than 2 bars shall be used in each rib with a minimum space between bars of 1 in. The width of ribs shall be 2 ins. plus the widths of the bars plus the spaces between the bars but the minimum width of rib shall be 4 ins.

The tile or blocks shall be regarded only as fillers, and shall not be considered in the design except as a dead load.

If designed as a T-beam, the slab portion above the fillers shall have a thickness of at least one-fifth the depth from the top surface to the centre of the steel and not less than 1½ ins., and shall consist of the same mixture used for the ribs, and shall be cast at the same time. Under these conditions it may be considered in the design of the ribs.

Tile or block fillers shall be laid with Portland cement mortar joints, and shall be thoroughly wet before the concrete is poured.

The protection for steel bars in the bottom of ribs shall be the same as for other beams.

To resist expansion and shrinkage stresses, reinforcement bars not less than ½ in. diameter shall be placed in the concrete at right angles to the ribs and above the fillers, at intervals not exceeding 30 inches.

Steel reinforcement shall have a minimum protection of concrete on all sides as follows:—

In columns and girders. 2 ins.; in beams and walls, 1½ ins.; and in floor slabs, 1 in.

The steel in footings for walls and columns shall have a minimum protection of 4 ins. of concrete.

All reinforcement shall be accurately located and mechanically secured against displacement during the placing of the concrete. Reinforcement bars for slabs shall not be spaced farther apart than two and one-half times the thickness of the slab. The spacing of parallel bars in beams shall be not less than two diameters from centre to centre, nor less than 1 inch. The clear spacing between two parallel layers of bars shall be not less than 1 in. In restrained or cantilever construction reinforcement shall extend beyond the supports into adjacent construction for full and effective anchorage, except that when this is not practicable, anchorage shall be obtained by other means acceptable to the (engineer or architect).

Special reinforcement shall be provided to resist concentrated loads.

Slabs reinforced in one direction only shall have shrinkage rods not less than ¼ in. in diameter placed above, and crossing the reinforcement and spaced not over 2 ft. apart.

All reinforcement shall be assembled well in advance of the placing of the concrete, and shall be inspected and approved by the (engineer or architect) before concrete is deposited.

Splices in reinforcing bars shall be designed to transfer the calculated stress at the joint either by bond and shear through the concrete or in vertical supports by bearing between the steel. Splices at points of maximum stress shall be avoided where possible. Lap splices of bars shall be of sufficient length to develop the required stress in the joint without exceeding the bond stress permitted, and the lapped portions shall be separated by a space not less than their thickness or a minimum of 1 in.

Members of web reinforcement in beams shall be designed for diagonal tensile stresses, using the calculated vertical shearing stress as a measure of these tensile stresses. The longitudinal spacing of vertical stirrups shall not exceed one-half the depth of beam and that of inclined members shall not exceed three-fourths of the depth of beam. It may be assumed that all of the external vertical shear in excess of the amount heretofore specified as allowable for concrete is provided for by the steel in vertical or inclined web members and bent-up bars.

Web members, such as stirrups, when not rigidly attached to the longitudinal steel at both top and bottom, shall be carried around and bent over the longitudinal members or otherwise sufficiently anchored in the compression concrete to develop the tensile stresses existing in them. Web members shall be rigidly attached to the longitudinal steel on the tension side. Stirrups at the ends of continuous girders shall be inverted with the free ends anchored in the compression concrete at the bottom of the beam. The length of stirrups or diagonals embedded in compression concrete shall be sufficient to develop their entire tensile stresses by adhesion.

Columns.—The length of columns shall be taken as the maximum unsupported length.

The unsupported length of columns shall not exceed 15 times the effective diameter or least effective thickness when longitudinally reinforced nor 10 times the effective diameter for hooped columns, and in no case shall the effective diameter or least effective thickness be less than 9 ins., measured within the ties or hooping from out to out of the longitudinal reinforcement. The length shall include any corbel or knee brace attached to the column. The effective diameter of columns supporting girdless floors shall be not less than one-twelfth the least panel dimension of panels supported by any column.

Bending stresses in columns due to eccentric loads shall be provided for by increasing the section of concrete or steel so that the total unit stress shall not exceed the allowable working stress in flexure.

Suitable steel base plates or castings shall be provided at the bottom of columns to distribute the loads over the footings, and the vertical reinforcement bars shall bear squarely on these plates or the reinforcing bars shall be carried down into an enlarged footing to distribute the load through bond stress.

In columns where necessary to splice vertical bars having areas in excess of 1¼ sq. ins., it shall be done by cutting the bars squarely at the ends and enclosing them in a close-fitting pipe sleeve, or uniting them by a threaded splice or other mechanical connection that will transfer the load from one to the other without stressing the adjoining concrete excessively. The middle point of such splices shall be within 1 ft. above the floor level. Splices in column hooping where necessary shall be sufficient to develop the full strength of the hooping.

All interior columns shall be round. When interior columns other than round are used for architectural effect, the corners shall be made of plaster on metal lath built around the columns after they are fully set.

Longitudinally reinforced columns shall have not less than 4 vertical bars or rods secured against lateral displacement by steel ties placed not farther apart than 16 diameters of the vertical bars or rods, nor more than 12 ins. The area of the vertical reinforcement shall be not less than 1 per cent. nor more than 4 per cent. calculated upon the effective area of the column, which is the area within the reinforcement. No vertical bars shall have a diameter or least dimension less than ⅝ in. and steel ties shall be not less than ¼ in. in diameter or least dimension.

Hooped columns shall be reinforced with vertical steel with an area of not less than 1 per cent. and not more than 4 per cent., calculated upon the effective area of the column. The vertical bars or rods shall be spaced not farther apart than one-eighth the circumference nor more than 8 ins., and be secured to the hooping. The reinforcement, in the form of hoops or spirals, shall be not less than 1 per cent. (that is, a volume of steel equal to the required per cent. of the volume of concrete within the hoops or spirals for a unit length of column), with a clear spacing not greater than one-sixth the effective diameter, and preferably not greater than one-tenth the effective diameter, nor more than 2½ ins. For hooped columns reinforced as herein specified, the unit concrete stresses may be 700 lbs. per square inch for 1:2:4 concrete and 870 lbs. per square inch for 1:1½:3 concrete.

The load which may be carried by any column shall be determined by the following formula:—

$$P = f_o (A_o + nA_s).$$

in which:

P = total load in pounds.
f_o = allowable compressive fibre stress in pounds per square inch on the concrete.
A_o = effective area of concrete in square inches.
n = ratio of deformation.
A_s = area of vertical steel in square inches.

Axial compression in structural steel columns thoroughly encased in concrete having a minimum thickness of 4 ins. and reinforced with not less than 1 per cent. of hooping steel (that is, a volume of steel equal to 1 per cent. of the volume of concrete within the hoops), equally divided between vertical reinforcement and spirals or hoops spaced not more than 12 ins. apart, may be taken at 16,000 lbs. per square inch on the net section of the structural steel, no allowance being made for the concrete casing. The spirals or hoops shall be placed not nearer than 1 in. from surface of the concrete. The ratio of length to least radius of gyration of the structural steel section shall not exceed 120.

Walls.—Exterior and interior bearing and non-bearing walls of reinforced concrete shall be securely anchored to all intersecting walls, columns and floors, and the allowable compressive stress shall not exceed 250 lbs. per square inch. The thickness shall be not less than two-thirds that specified for brick walls, and in no case less than 8 ins. All such walls shall be reinforced with steel running both horizontally and vertically. The amount of reinforcement shall be not less than one-fifth of 1 per cent. of the cross-section of the wall, and shall be equally disposed near each face of the wall; except that in walls or partitions 8 ins. or less in thickness, the reinforcement may be placed as a single layer in the middle. Reinforcement shall be spaced not more than 18 ins. apart. Additional reinforcement shall be placed around wall openings, and all vertical and horizontal reinforcement shall be wired or have other mechanical bond at intervals not exceeding 18 ins. in either direction.

General.—The 1916 Report of the Joint Committee on Concrete and Reinforced Concrete shall be followed in the design of girderless slabs, and it shall be the final authority in interpreting these specifications and applying same to unusual cases of spans and loading.

PRODUCTION OF SCHEELITE IN NEW ZEALAND.

Scheelite is found in several sections of New Zealand in the neighborhood of the gold mines both in the North and South Island, and has of late been quite extensively mined. The output has increased from 58 long tons, valued at $20,746, in 1909, to 130 tons, valued at $64,953, in 1912, and to 194 tons, valued at $135,211, in 1915.

Since September, 1915, the British government has requisitioned all supplies of scheelite and other ore containing tungstic acid, and from that date all exports to other markets have been prohibited. The price fixed by the Imperial Government was £2 15s. ($13.38) per unit (a unit being 1 per cent. of tungstic acid in the sample) delivered at London or Liverpool, the scheelite concentrate to contain not less than 65 per cent. (65 units of tungstic acid).

It was reported in the latter part of 1915 that a large body of scheelite was located in the eastern part of the North Island in the Hawke's Bay district.

DIAGRAMS FOR THERMAL CORRECTIONS FOR ROAD OIL AND FOR CONTENTS OF TANKS.

The specifications of the Road Department of Los Angeles County, California, for road oil for macadam contain the following clause for thermal corrections:—

For the purpose of measuring this oil, a temperature of 60° F. shall be deemed normal temperature.

The accompanying diagram was devised by E. Earl Glass, of the Road Department, for applying the thermal correction. The following example illustrates the method: A tank received measures 1,100 gallons of oil at 375° F.

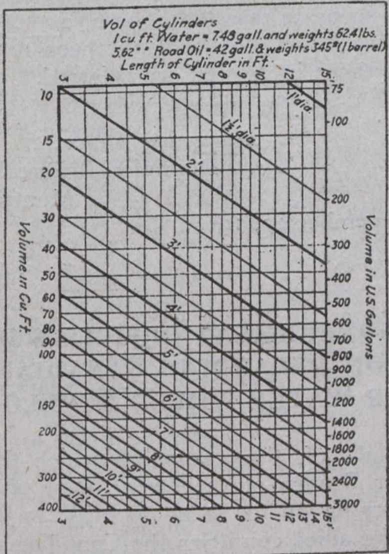


Diagram for Finding Contents of Tank.

The slant line 1,100 gallons hot oil intersects the 375° vertical line at 995 gallons, which would be the 60° F. volume of the load. The chart is used by the inspectors on road oiling.

The other diagram, also devised by Mr. Glass, is used for finding the contents of all ordinary sized tanks used in road oiling.

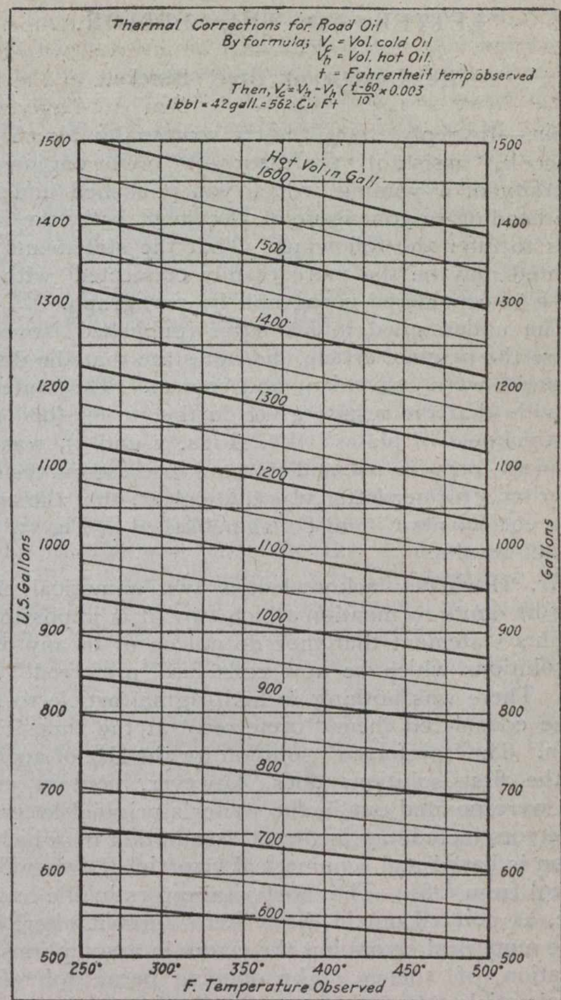


Diagram for Determining Thermal Corrections for Road Oil.

In determining the quantity of oil delivered, the correction for expansion by heat shall be as follows:—

From the measured volume of oil received at any temperature above 60° F. an amount equivalent to 0.3 of 1 per cent. for every 10° above said 60° F. shall be subtracted as the correction for expansion by heat.

JAPAN'S BOOM IN METALS.

The total value of the mineral output in Japan in 1915 was $87,711,075, exclusive of the production of the government iron works. This was a gain of $10,428,620 over 1914. The total value of metals produced amounted to $17,811,405, or 57 per cent. above the previous year's figures. The increase in copper output was 7 per cent., and in value more than 37 per cent. over 1914. Antimony output increased 200 per cent., while the gain in value was 8,750 per cent. Zinc showed an advance of 206 per cent. in output and 804 per cent. in value.

In Java Dutch government engineers have built a road bridge more than 100 feet long and with a central span of more than sixty feet entirely of bamboo.

The exports of coal from the United Kingdom in the 10 months ended October 31 last year were 32,741,158 tons, as compared with 36,944,758 tons in the first 10 months of 1915, and 52,060,846 tons in the first 10 months of 1914. These totals were increased to 35,150,172 tons, 38,830,606 tons and 54,523,993 tons by the addition of coke and patent fuel. The exports of bunker coal to October 31 were 10,960,984 tons, as compared with 11,745,472 tons and 16,037,409 tons. It will be seen that the coal exports have fallen off very decidedly when compared with 1914, and that a moderate further decline had occurred as compared with 1915.

FERRO-PRODUCTS.

Ferro-silicon and ferro-phosphorus were produced in Canada in electric smelting plants during 1915, the latter in small quantites only. Ferro-silicon, 50 per cent., 75 per cent., and 85 per cent., was made at Welland, Ont., by the Electro-Metals, Limited, and ferro-phosphorus at Buckingham, Que., by the Electric Reduction Co., Limited.

The total production of ferro-alloys during 1915 was 10,794 tons, valued at $753,404, as against a production of 7,524 tons, valued at $478,355 in 1914, and 8,075 tons, valued at $493,018 in 1913. In 1912 the production was 7,834 short tons, valued at $465,225, and in 1911, 7,507 short tons valued at $376,404.

The total number of locomotives ordered in the United States in the first half of last year was 2,124, while orders for 878 more were booked in the four months ending October 31. The year's aggregate was thus carried to 3,002 engines. The number of locomotives for which contracts were let in October was 386; of these 173 were ordered by foreign buyers.

$60,000,000 worth of construction during the next four years on extensions of the steel plant at Steelton, Pa., and Sparrows Point, Md., is announced by the Bethlehem Steel Co., which will build new blast furnaces, steel mills, and other important plant involving much construction work such as grading, foundations, steel-work, concrete.

LETTER TO THE EDITOR.

Re Design of Steel Stacks.

Sir,—In reply to a letter written by Mr. W. A. Hitchcock, assistant professor of civil engineering, University of Wyoming, which you published under the above heading in your issue of December 14th, the writer wishes to offer the following. That the statements here presented may be the more readily connected with the letter it is considered paragraph by paragraph.

The undersigned in his letter (published November 16th) re the original article, did not state that the designing curves were convenient and correct. The statement was made that the results given in the curves for "minimum thickness of plates" (i.e., Figs. 2 and 3), was in a convenient form for use and referred to these curves only. The writer's former letter was quite clear, that the curves for the coefficients C_1 and C_2, as published in Fig. 5, gave incorrect results.

Mr. Hitchcock acknowledges two numerical errors but as he omits to mention which two, it is impossible to check his statement that they do not occur in any of the first solutions which he now calls his "preferred" solutions. There was nothing in his original article to state that he considered them "preferred" at the time it was written. The "preferred" solution for the size of anchors, i.e., the first solution, does, however, contain errors which were pointed out in the writer's original letter and these errors include an incorrect assumption on which the solution is based and a numerical error in deriving Equation (29) from (28). The most vital errors in the original article, as pointed out in the writer's former letter, were not the numerical errors but the errors in assumptions and application of theory. The opening paragraph of the original article stated that some of the formulæ were new and that all diagrams were new. The undersigned did not, as Mr. Hitchcock would have us believe, state that nothing new had been given but that the article contained nothing new except the diagrams and the incorrect conclusions based on the errors. These incorrect assumptions may have "long been used by other designers," but this cannot be considered a reason for their preservation.

There was no intention to add another to the solutions given by anchor bolt sizes. The assumptions used in solutions (1) and (2) have been shown in the former letter to be untrue and results therefore incorrect. The assumption which the present writer substituted and which has not yet been proved wrong, shows, as was intended, that a solution along the lines followed in (1) and (2), if correct and if accurately applied, would give the same results as solution (3) when correctly calculated. Solution (3) should probably be considered the "preferred" solution, since it has the advantage of being based on a correct assumption, but even here, because of error in Equation (40), the result was incorrect. It cannot be considered a case of choosing, or trimming down, the design; the sizes used are chosen after the size, theoretically required, has been calculated. All theoretical solutions based on correct assumptions will, if accurately applied, give the same result.

The statement in the writer's letter regarding the comparison of solutions (1), (2) and (3) was not, as Mr. Hitchcock states, an error. There is, as was stated, no evidence in his article as to why he decides that solutions (1) and (3) give too high results and (2) too low, unless we consider the effect of the errors. If this is considered, the evidence is clear because the errors have had the effect of increasing sizes obtained by (1) and (3) and decreasing those derived in (2). This applies equally well to the specific problem of 12 bolts or to any other case.

The "apparent" error in Equation (44) is a real one. It is not a case for the designer to choose, but is the question of calculating correctly the stress per lineal inch along the circumference, at the base plate. The writer's Equation (44a) gives the correct result, while the original Equation (44) does not.

The author states that the fallacies of the original assumptions are admitted by all. If this had been stated in the original article or, better still, if the solutions based on these false assumptions had been omitted, the writer's original letter would have been unnecessary. It is not a question of the results being safe or unsafe but a question of obtaining accurate results. These, as set forth in the writer's former letter, have not yet been disproved, and to this letter those interested are referred.

H. M. WHITE,
Designing Office,
Dominion Bridge Co., Ltd.,
Montreal, Canada.

Montreal, December 27, 1916.

RUSSIAN GOVERNMENT OFFICIALS DESIRE INFORMATION REGARDING CANADIAN FACILITIES FOR WATER POWER DEVELOPMENT.

As a result of the campaign which the Dominion Water Power Branch of the Department of the Interior has been carrying on for some months, to make known throughout the allied countries the Canadian water power situation, there has been received an urgent request from Russia for some detailed information regarding facilities available in Canada which might be used in Russia for carrying out some very extensive hydro-electric power schemes now under consideration in that country.

The Dominion Water Power Branch has been asked to furnish information respecting Canadian engineering experience in the design and construction of hydro-electric developments, also regarding Canadian manufacturers of all kinds of equipment, electrical, hydro-electric and mechanical, required for a complete water power project. While every effort will be made by the Dominion Water Power Branch officials to secure this information direct, *The Canadian Engineer* has been asked to call the situation to the attention of its readers, in order that any one interested in furnishing the necessary information to Russia may be advised of the situation and communicate direct with the superintendent of that branch in Ottawa.

The secretary of the Canadian Society of Civil Engineers, Montreal, has been requested to furnish as complete a list as possible of the Canadian engineers who have had experience in the design and construction of our water power developments.

The iron ore exports from Algeria during the first nine months of the present year totalled 739,288 tons, as compared with 625,912 tons during the corresponding period of 1915.

Topographic surveys by the United States Geological Survey have now covered 41 per cent. of the entire area of the country, according to the annual report of Secretary Lane, of the Department of the Interior. During the year ending June 30, 1916, 22,715 square miles were mapped topographically, making the total area in the United States surveyed to date 1,237,520 square miles. Topographic surveys were made in 29 states, 17 of which co-operated in the work. In the Division of Water Resources 1,302 river gauging stations were maintained in 1916.

SLAB DEFLECTION AND SUBSIDENCE OF COLUMN SUPPORTS IN A FLOOR TEST OF INTERNATIONAL HALL, CHICAGO.*

By Henry T. Eddy, C.E., Ph.D., LL.D.,
University of Minnesota.

REINFORCED concrete floor slabs are somewhat imperfectly elastic, and in testing them an allowance is commonly made for this fact by the requirement that in case of any excessive deflection at least 80 per cent. of it shall disappear within a week after the removal of the load which produced the deflection. This requirement recognizes the imperfection of the elastic properties of the slab in two ways, since first, it does not forbid some residual permanent deflection, and, further, it does not require immediate recovery, neither of which concessions could be made in case of a perfectly elastic structure.

Few materials of construction are so perfectly elastic as actually to make an immediate and complete recovery, but a much larger margin is allowed to reinforced concrete than to most other materials. It should be noticed that on account of its being a composite structure, composed partly of steel and partly of concrete, of which the steel is more nearly perfectly elastic than the concrete, a floor slab with steel massed in the column heads will have a more prompt and complete recovery than one with so-called drop heads of concrete in place of the heavy reinforcement in the column heads.

But in the retardation or time lag of slab recovery a phenomenon is exhibited which is not found to any perceptible extent in other kinds of materials of construction in ordinary use. Since recovery from slab deflection is gradual and may never be complete, and since a corresponding gradual increase of deflection must occur under a load, it has been argued by some that final stability under a load is impossible. The permanence, however, of ancient concrete structures seems to show that there must be a limit to the deformations and deflections that will take place under loads, just as there is to the recovery after removal, and it must be admitted that concrete is not really plastic, although the property of concrete to which attention has just been directed bears some of the marks of plasticity. Were concrete really plastic under ordinary stresses, that fact would spell the ultimate destruction of buildings and bridges made of that material. But concrete is a kind of artificial rock and evidently reaches, in time, a permanent and invariable state in which progressive changes no longer take place, although, in course of hardening, phenomena may occur which, if not ultimately checked, would involve final collapse.

Now, the theory of the flexure of slabs differs from that of continuous beams in important particulars, one of which is that the applied moments and the observed resisting moments are not equal to each other, a fact the reasons for which are discussed elsewhere. But to this fact it is due that the economical and safe depth or thickness of slabs for any given span is small compared with that of beams, a fact which partially obviates one of the great drawbacks that exists to the use of continuous beams, *viz.*, the large stresses introduced by any accidental subsidence of supports.

As just stated, one of the most serious practical objections to the employment of continuous beams lies in the very large stresses induced by settlement of supports, because this will usually entirely disarrange the stresses for which the structure was designed. This hazard has operated very largely to prevent the construction of continuous bridges consisting of several successive spans. But a subsidence of a magnitude which might produce dangerous stresses in a structural steel building may be taken up in course of construction in a reinforced concrete building without requiring special attention, since the relatively large flexibility of these slabs is such as to make the effects of moderate subsidences inconsiderable compared with the effects of subsidences of the same absolute amount in bridges and deep girders.

The structural utility of continuous panels is so great as to outweigh their risks and practically to necessitate their adoption. The fact previously stated that the applied moments and resisting moments in slabs are unequal does not, however, prevent us from applying the theory of flexure to slabs as well as to beams, although it does prevent us from calculating the moment of inertia of the resisting materials in slabs in the same manner as in beams. But those deductions from the theory of flexure which are independent of the magnitude of the moment inertia are valid equally for beams and for slabs. Such, for example, are the relative deflections and vertical displacements of different points in the span. This it is which has made it possible to treat with success an important question which arose in the discussion of the test of the floor of International Hall.

In this floor slab the foundations of the columns proved insufficient to carry the test loads without yielding by amounts large enough to have considerable effects upon the stresses and deflections of the four loaded panels, each of which was 18 by 18 feet square. The floor as tested was a deck slab with no columns on its upper side. It was three panels wide and nine panels long, and was built into the surrounding walls of an old brick building by cutting into them somewhat for space to place steel and pour concrete, so that the edges of the slab were made integral so far as possible with the walls. The four panels which were tested formed a square near the middle of one side of the slab, two of them being in one tier of wall panels and the other two in the middle tier adjacent to them. In this floor, with its two rows of supporting columns parallel to the long sides of the building, certain vertical displacements were measured at the columns and at mid-span between columns and walls, both before and a week after the removal of the final test load, and the question was to determine whether the recovery was as much as 80 per cent. of the maximum deflection, and so whether there still remained as much as 20 per cent. of the maximum. The solution of this problem implicitly requires the determination of the magnitude of the vertical displacements which would occur in the slab by reason of the subsidences of the column supports only; for the actual vertical displacement at any point of the slab is the sum of two kinds of displacement; first, that due to the bending by reason of subsidence of the points of support, and, second, that due to bending by reason of the applied load. The fact that the subsidence of columns was itself also due to the applied load is immaterial, since the effect would be the same were the subsidence due to some other cause.

When that part of the vertical displacement at any point which is due to the subsidence of supports is subtracted from the actual vertical displacement observed at that point, the remainder is the true deflection and may be either that due to the effect of the load in bending the slab or to the residual bending effect after removing the load, according as the load is still resting upon the slab

*Journal of Franklin Institute.

or has been removed. The ratio of the remainder after removal to the remainder before removal will show how much the recovery falls short of being complete.

Let these four panels be designated by A, B, C, D, respectively, as shown in Fig. 1, with columns numbered from 1 to 6. Panel B is the centre of the entire floor. Let the reinforced direct belts extending across the slab over the columns be regarded as continuous beams fixed horizontally at the walls. The assumption that they are fixed at the walls is probably not entirely correct, but more nearly so than the assumption of mere support at the walls. It is found that the latter assumption gives results which do not differ much from those obtained by assuming the ends to be fixed in direction as well as position. If L be taken as the distance between successive columns, these belts or beam strips, which may be taken roughly as having a width of 0.5 L, are not of uniform moment of inertia throughout their lengths, but will, for the purpose of preliminary investigation, be assumed as uniform, and their irregularity of cross-section will be allowed for later.

Given a uniform continuous beam of three equal spans, each of length L, as shown in Fig. 2, whose extremities are fixed horizontally on the same level, but whose intermediate points of support at points 1 and 2 are depressed by the observed subsidence z_1 and z_2, respectively, below the level of the extremities, to find the displacements z_{12}, z_{01}, z_{23}, at the middle of the centre and end spans respectively, there being no stresses or displacements due to any other cause than the subsidences z_1 and z_2.

Let M_0 and M_3 be the moments at the extremities and M_1 and M_2 at the intermediate supports, respectively.

Then, by making use of the theorem of three moments, which applies in its general form to any two successive spans of a straight beam with supports at any arbitrary levels, we have the following four equations by taking four successive pairs of spans, as follows: The first pair, Equation (1), consists of a span of zero length and a span extending from the wall at 0 to the first row of columns

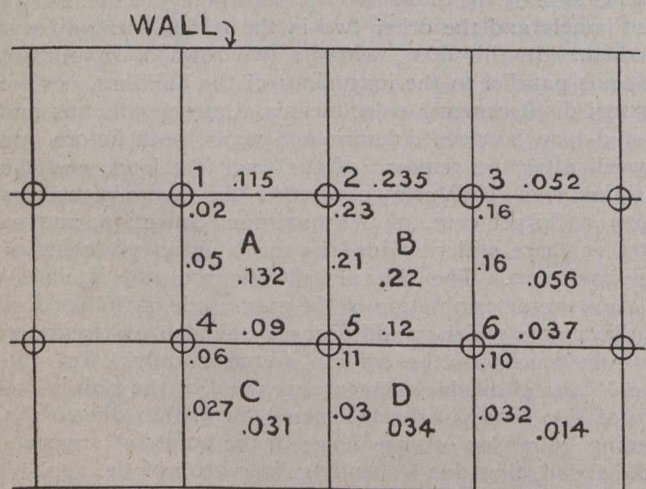


Fig. 1.

at 1. The effect of a span of zero length is to give the slab a fixed horizontal direction at the wall. The second pair of spans, Equation (2), extends from the wall at 0 to the first row of columns and from the first to the second row of columns at 2. The third pair, Equation (3), extends from the first row of columns at 1, across the second row to the other wall at 3, and the fourth pair, Equation (4), from the second row of columns at 2, to the second wall at 3, and includes a span of zero length at that wall.

The notation may be understood from Fig. 2, in which the tangents at the 0 and 3 are fixed in a horizontal position and the vertical displacements or subsidences at

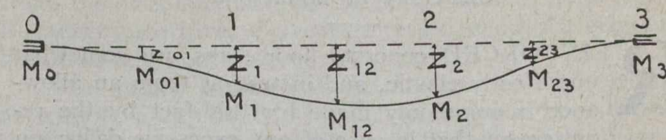


Fig. 2.

the intermediate supports 1 and 2 are of known amounts, z_1 and z_2, and the vertical displacements at mid-span are designated by z_{01}, z_{12}, and z_{23}, respectively, while the applied bending moments are denoted by the letter M with corresponding subscripts, and the length of the equal spans between supports by L.

$$6\,EIz_1 = (2M_0 + M_1)\,L^2 \quad \ldots\ldots (1)$$
$$-6\,EI\,(2z_1 - z_2) = (M_0 + 4M_1 + M_2)\,L^2 \quad \ldots\ldots (2)$$
$$6\,EI\,(z_1 - 2z_2) = (M_1 + 4M_2 + M_3)\,L^2 \quad \ldots\ldots (3)$$
$$6\,EIz_2 = (M_2 + 2M_3)\,L^2 \quad \ldots\ldots (4)$$

Considering now the mid-displacement z_{12} of the middle span 12, it may be expressed in terms of the two end moments and end displacements of that span as follows:—

$$6\,EI\,(z_1 + z_2 - 2z_{12}) = \tfrac{3}{4}\,(M_1 + M_2)\,L^2 \quad \ldots\ldots (5)$$

as appears from the fundamental equation of moments and shears used in establishing the equation of three moments.

The solution of these five simultaneous linear equations by the method of indeterminate multipliers or otherwise gives as a result

$$z_{12} = 5/8\,(z_1 + z_2) \quad \ldots\ldots (6)$$

A similar consideration of the mid-displacement z_{23} of the end span 23 shows that

$$6\,EIz_{23} = 1/8\,(M_2 + 5M_3)\,L^2 \quad \ldots\ldots (7)$$

The solution of the first four simultaneous linear equations and (7) gives

$$z_{23} = (19z_2 - 4z_1)/40 \quad \ldots\ldots (8)$$

and by symmetry

$$z_{01} = (19z_1 - 4z_2)/40 \quad \ldots\ldots (9)$$

Hence

$$z_{01} + z_{12} + z_{23} = z_1 + z_2 \quad \ldots\ldots (10)$$

which is a fundamental equation of displacements in this case of no loads.

In case the assumption were that the walls are simple supports and exert no restraint, the values of the displacements at mid-span have been found to be

$$\left.\begin{aligned} z_{12} &= 23\,(z_1 + z_2)/40 \\ z_{23} &= (29z_2 - 6z_1)/40 \\ z_{01} &= (29z_1 - 6z_2)/40 \end{aligned}\right\} \quad \ldots\ldots (11)$$

from which it appears that this would make the displacement at mid-span of the middle span a little less and at mid-span of the end spans a little greater than when the ends are fixed, and the same kind of effect, but of smaller amount, will be produced by any relaxation of fixity at the walls.

It will be noticed from (11) that

$$z_{12} = z_{01} + z_{23}$$

It is evident that the floor under consideration may be taken to consist of beam strips not only crosswise from side wall to side wall, as has just been done, but also as made up of beam strips parallel to the side walls and extending lengthwise of the building, and that the agreement of the results of computations by these two methods will tend to establish their correctness. Suppose the beam strips to each have a width of $\frac{1}{2}L$ extending over the two rows of columns parallel to the long side walls, with a strip of the same width lying between these two. If the length of these strips be taken as $4L$, they may be assumed to have their ends practically fixed horizontally, with certain observed vertical displacements at the three intervening columns. The problem in this case is this: Given the vertical displacements z_1, z_2, and z_3 at points of support 1, 2, and 3 of a beam fixed horizontally at the ends 0 and 4, to find the vertical displacements z_{01}, z_{12}, z_{23}, and z_{34}, at mid-span between each pair of supports. The equations are like those already used in the case of three spans, and the values arrived at in the same manner are:—

$$\left.\begin{aligned} z_{01} &= (106z_1 - 21z_2 + 6z_3)/224 \\ z_{12} &= (142z_1 + 133z_2 - 30z_3)/224 \\ z_{23} &= (142z_3 + 133z_2 - 30z_1)/224 \\ z_{34} &= (106z_3 - 21z_2 + 6z_1)/224 \end{aligned}\right\} \ldots\ldots\ldots (12)$$

It will be noticed that in this case

$$z_{01} + z_{12} + z_{23} + z_{34} = z_1 + z_2 + z_3 \ldots\ldots (13)$$

which is an equation of deflections for four spans similar to (10) for three spans.

On Fig. 1, which shows a plan of the panels of the test, are inscribed, at columns 1 to 6, the amounts of the vertical displacements observed under the final load in inches, and at mid-span between these columns the vertical displacements, computed according to the foregoing equations, that would take place in a uniform slab by reason of the displacements at the columns without applying any load to the slab except the reactions at the columns which are necessary to produce the displacements at the columns.

Displacements at panel centres are also given. These last are computed both by taking beam strips crosswise and lengthwise of the floor midway between the beam strips already computed across the tops of the columns. Practically the same values at panel centres are obtained from the computation by strips crosswise as lengthwise. This affords a satisfactory check on the work. Were the slab of uniform moment of inertia throughout, the displacements which have been computed at mid-spans and panel centres would express the position of the surface from which true deflections should be measured, or the surface of zero deflection from which deflections due to the load or lack of recovery are to be measured. That, however, is not entirely correct for a floor slab, by reason of the great relative stiffness and extent of the column heads, which must be allowed for. The side belts are weaker near mid-span than elsewhere. The kind of effect that this produces may be made evident by considering what would be the effect of joints at mid-span of the beam strips across the floor. That would cause angles in the strips at each mid-span with comparatively straight portions over the columns. At mid-span in the centre of the strip the displacement would evidently be increased by such joints or by any weakness of this kind. In the case of the test under consideration it is estimated that the combination of a central conduit in each panel tier parallel to the long side walls, with the lack of columns integral with the slab above it, added not less than 25 per cent. to the computed vertical displacements at the centres of panels *A* and *B* and midway between those points, these being the points of maximum deflection at which the percentage of recovery is to be determined.

The accompanying table gives in column 1 the computed vertical displacements at these points, and in column 2 these amounts increased by 25 per cent., to take account of the increased displacement by reason of lack of uniformity in the slab. Column 3 gives the observed displacements under maximum load, and column 4 the observed displacement after removal of the load and recovery. Column 5 is the difference between column 3 and column 2, or the deflection under the load. Column 6 is the difference between column 4 and column 2, or the deflection after removal of load and recovery. Column 7 is the ratio of column 6 to column 5, or the percentage which the residual deflection is of the maximum. Column 8 is 100 per cent. less the per cent. in column 7, or the percentage of recovery, which has a mean value of 80.7.

Table of Vertical Displacements and Deflections in Inches.

At centre of	1 Computed for uniform slab.	2 Increased 25 per cent. not uniform.	3 Maximum observed displacement.	4 Minimum observed displacement.	5 Maximum deflection column 3 less column 2.	6 Minimum deflection column 4 less column 2.	7 Percentage of residual deflection.	8 Percentage of recovery.
Panel A..........	0.13	0.16	1.04	0.38	0.88	0.22	25	75
Span 2—5........	.21	.25	.95	.37	.70	.12	17	85
Panel B..........	.22	.264	.98	.38	.716	.116	16	84
Mean	0.19	0.225	0.99	0.38	0.765	0.152	19.3	80.7

The application of a rule requiring a recovery of 80 per cent. in this case, however, discriminates against the floor structure and attempts to have it make good the shortcomings of the faulty foundations on which the columns rest. At the time of the maximum load, when subsidence of the columns took place, the columns were subjected to a very considerable bending moment by reason of the tipping of the floor, which was integral with the column heads. This caused the foundation of each column not only to sink vertically, but to tilt at the same time in an inelastic displacement. On removal of the load, any restoration of the foundation either to its original vertical position or level would have to be accomplished, not by any elastic effort of the foundation itself, but by an elastic effort coming from the floor itself. Not only was work required to displace the foundation at first, but additional work will be required after the removal of the load to right the foundation, or else the foundation will itself tend to restrain the slab and hold it in its displaced position not merely by reason of its vertical displacement, but by reason of its tilted position in addition to that. Thus it is that the displaced foundations tend to prevent the recovery of the slab in other ways than merely by the vertical displacements which have been considered in the preceding computations.

CUBAN MANGANESE INDUSTRY.

The United States Consul reports that the manganese industry of Cuba, which dates back about twenty years, has been continued, with certain interruptions, up to the present time. There are within the Santiago district three generally recognized deposits of ore known as Cristo, Cauto and Ponupo, the last-mentioned being the largest, and having produced up to the present about 2,000,000 tons. It is still producing about 3,000 tons per month of a fair grade of furnace ore averaging about 43 per cent. metallic content. The Cristo group is producing a small amount of ore that runs about 46 per cent., while the Cauto group is producing about 2,000 tons per month. During 1915 practically all the ore produced was shipped to Baltimore, with the exception of a small consignment to Italy. The demand for furnace ore is still strong. None of the ore is treated locally, except by washing to remove the dirt. Large deposits of manganese still remain undeveloped, no particular attention having been given to them.

TRACTIVE RESISTANCE TESTS WITH AN ELECTRIC MOTOR TRUCK.

THE Research Division of the Electrical Engineering Department of the Massachusetts Institute of Technology during 1915 conducted an investigation of tractive resistance of urban roads to a motor delivery wagon equipped with solid rubber tires. The results of the tests are described in a recently issued bulletin, from which the matter that follows is taken.

The subject of the research was to determine the resistance, including air resistance, offered to an electric truck, by level urban roads of different surface varieties, at standard truck speeds not exceeding 25 km. (15.5 miles) per hour. For this purpose, the output of the storage battery on a test truck was measured, for both directions of travel, over standard road beds, at different controller speeds. From this output were deducted all the corresponding electrical and mechanical losses in the truck mechanism, as determined by laboratory tests. The remainder of the output was consequently attributed to (1)

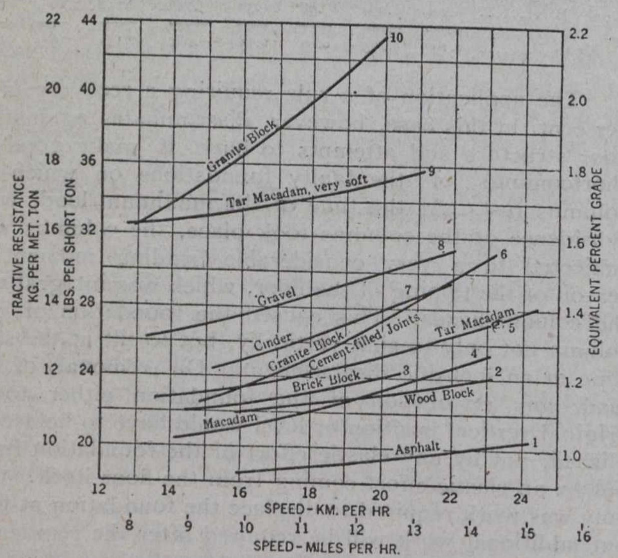


Fig. 1.—Summary of Tractive Resistance Tests.

road- (2) air- and (3) wind-resistance. The wind resistance was eliminated by averaging the results for both directions of running, leaving as the final result the sum of the road and air resistances.

By "road resistance" is meant the horizontal force required to pull the truck, assumed as internally frictionless, over the horizontal road in the absence of air. By "air resistance" is meant the horizontal force on the truck required to overcome the resistance of the air, assumed as quiescent in the absence of the truck. By "wind resistance" is meant the horizontal force on the truck necessary to overcome the resistance of the wind velocity, or that velocity of the air with respect to the ground which exists in the absence of the truck.

A 1,000-lb. (450-kg.) worm-drive, single-reduction electric truck, or delivery wagon, was placed at the disposal of the Research Division for the purpose of test. Its specifications are as follow:

Load capacity, 1,000 lbs. (450 kg.), equipped with one d.c. series motor; overall length of frame, 4,280 mm. 168½ ins.; width of frame, 890 mm. 35 ins.; wheel base (i.e., distance between centres of front and rear wheels, when front and rear axles are parallel), 2,730 mm. 107⅝ ins.; wheel gauge, 1,470 mm. 58 ins.

The total weight of the truck, including motor, battery and body, but without load or passengers, was 4,200 lbs. (1,910 kg.). Each of the four wheels was equipped with one solid rubber demountable tire (manufactured for this type of delivery wagon) rated at 36 ins. by 2½ ins. (91.5 c.m. by 6.35 c.m.), and actually measuring about 35 ins. (89 c.m.) tread diameter, and 2½ ins. (6.35 c.m.) width of base. The brakes were of the internal expanding type on each rear wheel.

Typical results for all classes of urban roads tested are summarized graphically in Fig. 1, and numerically in Table I. It appears that there are three principal elements which determine the tractive-resistance-speed curve for unit weight of a given vehicle, within the range of conditions covered by this test:

(1) A constant resistance, see curve 1, Fig. 2; the magnitude A of which depends on the lack of resilience of the road surface and wheel tire material, i.e., on the energy losses due to displacement of tire material and road-surface material. This constant element A would be encountered upon a smooth level road of the particular type considered, in the absence of impact, air, and wind resistances.

(2) An increasing resistance with increasing speed, due to impact losses (curve 2), which results from lack of smoothness of road surface; losses of this nature are usually known to vary approximately as the second power of the velocity at impact; and

(3) An increasing resistance with increased speed, due to air pressure against the front of the vehicle, curve 3, which resistance is known to depend, roughly, on the second power of the speed. The sum of the three curves for items 1, 2 and 3, for the case of asphalt roads, results in curve 4. The constant resistance (1) may be briefly called the displacement resistance, item 2 the impact resistance, and item 3 the air resistance. The displacement resistance is low for hard pavements and high for soft pavements (of low resilience). The impact resistance is very marked in granite-block roads, as already mentioned. The air resistance, at any definite velocity, is the same for all curves; because the air-resisting parts of the truck were left unchanged throughout the tests. For an asphalt road in poor condition, at a speed of 20 km. per hour (12.4 miles per hour) the displacement resistance is 0.84 per cent., the air resistance is 0.11 per cent., the impact resistance 0.20 per cent. and the total 1.15 per cent. equivalent grade.

The displacement resistance of a road manifestly varies, not only with the type and surface quality of the road, but also with the type, dimensions and quality of the tires on the wheels of the vehicle. In the tests here reported, the same tires were used throughout, and they remained in substantially the same condition.

The impact resistance of a road manifestly depends not only on the type and surface quality of the road, and the sizes of its irregularities; but also on the type, dimensions and quality of the wheel tires, the weight of the truck, and the quality of its springs.

The air resistance per unit weight of truck manifestly depends upon the weight, dimensions and shape of the vehicle, as well as on the speed of the vehicle relatively to the surrounding air.

The wind resistance per unit weight of truck manifestly depends upon the weight, dimensions and shape of the vehicle, as well as on the direction and velocity of the wind and the velocity of the vehicle. It is assumed that at low wind and vehicle speeds, like those here considered, only that component of the wind which is in the direction of the vehicle's path needs to be taken into account, and that the mean of the wind resistance in opposite directions, along the road, is zero.

Summary of Conclusions.—The following conclusions are indicated from the preceding results: as confined to urban roads, with a solid rubber-tired motor truck between the speed limits of from 13 to 25 kilometers per hour (8 to 15.5 miles per hour).

(1) The over-all efficiency of the test-truck mechanism, as described in this report, between battery terminals and rear-wheel treads, reached a maximum value of about 78 per cent., under the most favorable conditions.

(2) The mechanical efficiency of transmission from motor shaft to rear-wheel treads, for the truck tested, shaft-driven through a single-reduction worm gear, was found as high as 90 per cent.

(3) Tractive resistances are most conveniently expressed as an equivalent percentage grade; i.e., a level road of definite tractive resistance may be regarded as a road of zero tractive resistance, but rising uniformly x units in 100 units of road length, or having an equivalent grade of x per cent.

(4) Under the conditions of these tests, the tractive resistance on level roads, in the absence of wind, is composed of (a) displacement resistance, (b) impact resistance, and (c) air resistance.

(5) The displacement resistance varied from 0.85 per cent. equivalent grade, for a hard, smooth asphalt or bituminous concrete to 1.6 per cent. for a very soft tar-macadam road, and was practically constant, for all speeds considered, on any given road.

(6) The impact resistance increases with the velocity, with the total weight of vehicle, and with increasing road-surface roughness. In these tests, the impact resistance of good asphalt or bitulithic or other smooth pavement, was practically negligible, and reached its highest values on granite-block roads with sand-filled joints, and on badly worn macadam pavements. The rate of increase of impact resistance with speed was most marked on the roughest roads.

(7) At the vehicle speed of 20 km. (12.4 miles) per hour, the air resistance for the vehicle tested, assumed to be dependent only on the speed, was roughly 0.11 per cent. equivalent grade; i.e., from 4 per cent. of the highest, to 12.5 per cent. of the lowest, total tractive resistance.

(8) The following urban pavements are numerated in the order of their desirability for vehicle operation from the point of view of tractive resistance at 20 km. (12.4 miles) per hour, as found in this investigation: (1) asphalt, (2) wood block, (3) hard, smooth macadam, (4) brick block, (5) granite block with cement-filled joints, (6) cinder, (7) gravel, (8) granite block with sand-filled joints.

(9) The equivalent grade at 20 km. (12.4 miles) per hour of a badly worn city macadam road, was found to be nearly three times as great as that of the best asphalt

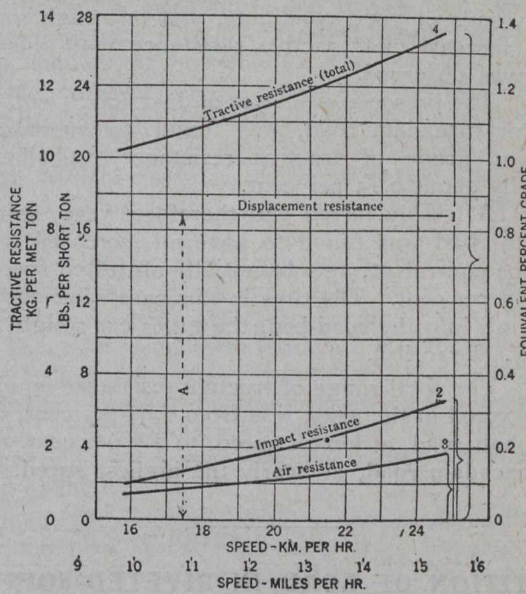


Fig. 2.—Approximate Analysis of Typical Tractive Resistance Into Its Elements for Asphalt Road in Poor Condition.

Table I.—Summary of Tractive Resistances of Different Urban Roads at Different Speeds.

All tractive resistances are expressed in equivalent per cent. grades.

Road. Type.	Condition.	Equivalent per cent grade. at 16 km. per hour (10 miles per hour).	Equivalent per cent grade. at 20 km. per hour (12.4 miles per hour).	Per cent. increase in tractive resistance from 16 to 20 km. per hour.	Comparative tractive resistance factors referred to asphalt roads. at 16 km. per hour.	Comparative tractive resistance factors referred to asphalt roads. at 20 km. per hour.
Asphalt	good	0.93	0.97	4	1.0	1.0
Asphalt	poor	1.03	1.16	11	1.11	1.20
Wood block	good	1.10	1.15	5	1.18	1.18
Brick block	good	1.12	1.21	8	1.20	1.25
Brick block	slightly worn	1.14	1.27	11	1.23	1.31
Granite block	good	1.83	2.16	18	1.97	2.23
Granite block with cement joints	good	1.16	1.37	18	1.25	1.41
Macadam, water bonded	dry and hard	1.06	1.17	10	1.14	1.20
Macadam, water bonded	fair, heavily oiled	1.63	1.76	8	1.75	1.82
Macadam, water bonded	poor, damp some holes	1.65	1.89	15	1.78	1.95
Tar macadam	good	1.17	1.27	9	1.26	1.31
Tar macadam	very soft	1.67	1.76	5	1.80	1.81
Tar macadam	many holes, extremely poor, soft	2.38	2.75	16	2.55	2.85
Cinder	fair, hard	1.25	1.39	11	1.35	1.43
Gravel	fair, dusty	1.37	1.50	9	1.47	1.55

road tested. This means, at this speed, a consumption of energy at wheel treads, of nearly three times as much on level poor macadam roads as on good level asphalt roads.

(10) Increasing the gross weight of the vehicle by 12 per cent., through load, was found to have no effect on tractive resistance within the observed speed limits for smooth roads in good condition; but on rough roads, a distinct increase in tractive resistance with this extra weight was observed.

(11) The presence of a layer of dust, say, 1 cm. thick, on a fair macadam road, was found to increase the equivalent grade of tractive resistance, at all tested speeds, by about 0.15 per cent.

(12) A freshly tarred and therefore very soft tar-macadam road was found to have an increased tractive resistance equivalent, at substantially all tested speeds, of about 0.5 per cent. The tires in this case sank about 0.8 in. (2 c.m.) into the road-bed, the gross car weight being 2,140 kg. (4,710 lb.).

(13) The total range of tractive resistance equivalent grade covered in the tests, was from 0.93 per cent, on the best asphalt road, at lowest speed, to 2.7 per cent. on the worst macadam road, at nearly the highest speed.

PARTITION OF LOAD IN RIVETED JOINTS.

IN the November issue of the Journal of the Franklin Institute an article appeared written by Cyrol M. Batho, assistant professor of applied mechanics at McGill University, in which he described the theoretical and experimental work carried on there in connection with ascertaining the load carried by each rivet in any form of riveted joint. Experiments were made upon different forms of joints, having a single line of five rivets on each side of the joint, and loaded in tension. The analyses indicated that a series of equations can be obtained giving the load carried by each rivet in any form of riveted joint in terms of a quantity K, which if the results are in shear depends upon the manner in which work is stored in the rivets; or if they act by frictional hold on the plates depends upon the work stored in the parts of the plates so held.

Only the specimen with ½-in. rivets was carried beyond the working load, but the regularity of its action showed that the partition of load obeyed the same laws at all loads up to that causing permanent deformation of the plates or rivets. In every specimen and at all loads the first and fifth rivets took by far the greater part of the total load, the actual proportion decreasing gradually as the load increased. For example, in the specimen with ½-in. rivets, the first and fifth rivets carried 83.5 per cent. of the total load at a load of 10,000 lbs., and this decreased to 70.0 per cent. at a load of 30,000 lbs. The latter load corresponds to an average stress of about 12,650 lbs. per square inch of actual rivet section, or 15,280 lbs. per square inch of nominal rivet section. This would usually be taken as the shearing stress on all the rivets, but actually the end rivets, if, as there seems little doubt, the rivets were in shear and not holding by friction, were each under an average shear stress of 22,150 lbs. per square inch, while the third rivet at the same load took only 3.2 per cent., corresponding to an average shear stress of only 2,020 lbs. per square inch. Thus in joints having several rows each containing an equal number of rivets and designed in the usual manner, i.e., allowing the average load per square inch of rivet section to be equal to the working stress in shear of the rivet material, the rivets in the end rows must carry stresses far above the allowable working stress.

The above refers to the specimens in which the cover plates were of the correct thickness; i.e., each half as thick as the middle plate. If they are thicker, the first rivet takes an even greater proportion of the load, the proportion increasing with increased thickness. In specimen B, in which the cover plates were of correct thickness, the first and fifth rivets took 88.1 per cent. of the load at a load of 16,000 lbs. In specimens D, E and F at the same load they carried 86.2 per cent., 89.7 per cent. and 78.9 per cent. of the total load respectively, but of these the first rivets carried respectively 70.7 per cent., 63.8 per cent. and 50 per cent. of the total load. Specimen F, in which the middle plate was of varying width, illustrated the action in members riveted to a gusset plate, and it was found that the varying width of plate resulted in a rather more even distribution of stress, the first and fifth rivets carrying only 69.7 per cent. of the load, as compared with 78.9 per cent. when the middle plate was cut down to uniform width.

In all the specimens tested the ratio of width of cover plate to pitch of rivets was the same, ¾. K probably varies as the width of cover plate divided by the pitch of the rivets; thus, with a smaller pitch or wider plates, K would be increased, and the effect of this would be to make the partition of the load rather more uniform. But a large variation of K only causes a comparatively small alteration in the percentage of the load carried by the end rivets. For example, in the specimen A a change of K from 0.485 to 1.3 only altered the load carried by each of these rivets from 40.7 per cent. to 35 per cent., and the alteration for a given change becomes less and less as the values of K increase. Thus the effect of change of pitch or breadth of cover is not likely to be very great, except possibly in splices containing a number of rivets in each row. However, further experiments are needed in order that a general law may be found for the value of K. When this is determined it will be possible to predetermine the exact partition of load in any proposed joint, and this will enable the joint to be designed in the most efficient manner. A very good approximation, sufficient for most purposes, may, however, be obtained from the data already given, since the general manner of partition of load is the same for all values of K.

Prof. Batho's summary and conclusions are as follow:—

1. It is shown that a riveted joint may be considered as a statically indeterminate structure, and that a series of equations may be obtained for any joint by means of Principle of Least Work, giving the loads carried by each of the rivets in terms of a quantity K, which depends upon the manner in which work is stored in, or by the action of, the rivets.

2. This theory is applied to various types of joints, and the modifying effects of non-uniform distribution of stress in the plates, unequal partition of the load between the two cover plates, and a difference in the modulus of elasticity of the middle plate and the cover plates are also considered.

3. It is shown experimentally that extensometer measurements on the outer surfaces of the cover plates of a riveted joint are sufficient for the determination of the mean stresses in the plates, and that the partition of the load among the rivets may be determined from such measurements. It is also shown that, at any rate after the first few loadings, the distribution of strain in the plates of a joint is not altered by repeated loadings.

4. It appears from 3 that if there is any frictional hold between the plates, it acts only over those portions in the immediate neighborhood of the rivets. All the experiments tend to show that friction does not play an imporant part, but further experiments are necessary on this point.

5. Experiments made on a number of specimens having a single line of rivets and loaded in tension give results in close agreement with the theoretical considerations. They also show that the longitudinal stresses in a portion of the cover plate between two consecutive rivets are a minimum along the line of rivets, rising to a maximum at the edges of the plates.

6. The experiments show that the value of K for a joint having a given ratio of width of cover plate to rivet pitch and a given number of rivets varies approximately directly as the load and inversely as the area of the rivets. An empirical rule is given for its value in joints similar to the experimental specimens, but a more general rule cannot be given until further experiments have been made. A theoretical estimate is made of the value of K for a rivet acting in shear, and the result is shown to be within the range of the experimental values.

7. Both the experimental results and the theoretical deductions show that: (a) in a double-cover butt joint having a single line of rivets, the two end rivets and the two rivets on each side of the junction of the middle plates take by far the greater part of the load at all loads within that causing permanent deformation of the plates or rivets, the actual proportion decreasing slowly as the load increases; (b) if, in such joints, the total area of cross-section of the cover plates is equal to that of the middle plates, these four rivets take equal loads, but if it is greater the end rivets take greater loads than the others, the difference increasing as the area of the cross-section of the cover plates increases; (c) if two plates of uniform width and equal thickness are connected by a single line of rivets to opposite sides of a gusset plate of uniform width, the first and last rivets take the greater part of the load, but if the gusset plate increases in width from the first to the last rivets, the partition of load is more uniform.

WORLD'S OUTPUT OF GRAPHITE.

Interest in graphite at present is widespread. According to the Canadian Mining Institute Bulletin the two principal forms in which the mineral is found are amorphous and crystalline, the former being very common. The world's production statistics (in tons) for 1912, the latest available, are as follows: Ceylon, 36,660; Canada, 2,060; South Africa, 42; Austria, 50,017; Madagascar, 3,011 United States, 3,835; Mexico, 3,158; Korea and Japan, 8,363; Germany, 13,814; Italy, 14,517; Sweden, 87; Norway, 285; France, 661; total, 136,510.

PAVING EQUIPMENT FOR CUBA.

One of the largest single shipments of paving equipment on record is that just made to Messrs. Torrance and Portal, of Havana, Cuba, who have secured extensive paving contracts for Havana and Cienfuegos.

The shipment made by the Iroquois Works of the Barber Asphalt Paving Company consisted of six cars routed by way of Key West, Florida, carrying two three-unit asphalt plants, two 2,000-gallon and one 1,000-gallon steam-heated melting kettles, four tandem rollers, two portable boilers and engines, fire wagons, paving tools, etc.

Contracts already secured by the Havana firm for paving in which Trinidad lake asphalt will be used total about 400,000 square yards. One of the Iroquois plants is to be located at Havana and the other at Cienfuegos.

BITUMINOUS SAND-GROUT PAVEMENT.

J. A. BRODIE, city engineer of Liverpool, Eng., has developed a type of pavement which has not yet been constructed in this country, but which is most interesting from the standpoint of a low-first-cost country or interurban road. Mr. Brodie calls his pavement "pitch macadam," but it has been referred to by most engineers who have reported upon it as a "bituminous sand-grout pavement." He has laid forty-one miles of it in Liverpool, and some of his roads have been in service for over fourteen years and are still in good condition.

Col. Wm. D. Sohier, chairman of the Massachusetts Highway Commission, has laid three experimental stretches of road following Mr. Brodie's ideas, although altering his specification somewhat in order better to suit local conditions. About 300 ft. of these experimental roads was laid with asphalt, the remainder being laid with tar. These Massachusetts roads are less than three years old, so that it is a little early to predict their success, but Col. Sohier says that there is no question in his own mind but that the bituminous sand-grout pavement will be stronger and wear longer than any other bituminous penetration or mixed road.

In all, 381 miles, or 44,532 square yards, of this pavement have been laid in Massachusetts. The engineers in charge of the work report that its present condition is excellent, and that there have been practically no repairs. All three sections are on main routes; motor vehicle traffic being probably 90 per cent. of the total, with considerable heavy motor trucking and teaming.

Owing to the success of these roads in Liverpool, there is no doubt but that some enterprising municipality or contractor will experiment with them in Canada in the near future. The following is a résumé of Mr. Brodie's specifications:—

The Brodie Specification.—"Upon a 10-inch hand-pitched foundation, laid and consolidated in accordance with the specification for water-bound macadam, a layer of dry macadam of 2½-inch gauge stones, similar in quality to that used for water-bound macadam, shall be spread evenly to a depth of about 3½ inches (before consolidation). This layer, after being rolled with a light steam roller, shall be grouted with a hot mixture of pitch and creosote oil prepared in accordance with the specification below, and again rolled while hot until the mass is thoroughly consolidated. A second layer of similar macadam of 1½-inch gauge stones 3 inches deep shall then be laid, preferably while the lower layer is still hot. After being rolled dry it shall be grouted in a similar manner with the pitch mixture, and again rolled until the whole is consolidated. The surface shall be finished off with a sprinkling of dry granite chippings. The road shall be laid to an approximately circular camber, with a cross-fall from crown to channel of 1 in 48.

"The pitch mixture used for grouting shall consist of coal tar pitch and creosote oil supplied to the specifications given below, and boiled together in a tank in the proportions of approximately 70 gallons of oil to one ton of pitch, a temperature of from 250° to 300° F. being attained. The proper consistency shall be obtained by applying the following rough tests, *viz.*:—

"A small sample of the mixture when cooled in water to 60° F. shall stretch at least 3 feet without breaking, the threads pulling out very fine. It shall also, when doubled into length of about one foot, bear hitting hard on a hard surface without showing any signs of cracking.

"After leaving the boiler and immediately before being put on the road, an equal quantity of fine shore sand heated to 400° F. shall be added to the pitch mixture, which shall then be kept continually stirred until spread.

"The macadam shall be of durable granite or trap rock from the quarries of North Wales or from other approved quarries having a similar class of rock. It shall be carefully broken into cubical form so as to be capable of passing through the specified gauge in any direction. It shall be cleanly riddled to free it from dust and all flat slatey fragments shall be picked out before shipment.

"2½-inch macadam shall pass through a 2½-inch ring and be held by a 2-inch ring.

"1½-inch macadam shall pass through a 1½-inch ring and be held by a 1-inch ring.

"The pitch shall yield no matter volatile below 270° C. when subjected to dry distillation, and its total volatile organic matter shall not fall below 30 per cent.

"It shall not contain more than 80 per cent. of its weight of matter insoluble in petroleum spirit of 0.700 specific gravity (boiling) and must be free from extraneous matter, such as sand and grit.

"It must twist fairly after immersion for two minutes in water at 60° C. but not under 550° C. (Hereinafter referred to as Clause 3.)

"The oil supplied shall be obtained exclusively by the distillation of coal tar, and shall not contain any portion of the distillate obtained below 240° C. None of it shall re-distil below 240° C.

"The oil as obtained by distillation of coal tar shall not be treated in any way, either by the addition of any coal tar product, or by the extraction of any of its constituents, excepting such extraction as may be necessary to comply with clause 3. It shall contain no moisture. It shall contain no solid matter at 15° and shall have a specific gravity of not less than 1.075 (taking water as 1.00 at 15° C.).

"It shall contain not less than 40 per cent. of its constituents that do not distil over below 320° C. and the 60 per cent. which does distil over below 320° C. shall contain 10 per cent. of tar acids, to be extracted by soda of specific gravity 1.125 (water 1.00)."

The Massachusetts Type.—The three Massachusetts bituminous sand-grout pavements have been constructed by A. W. Dean, chief engineer, and F. C. Pillsbury, division engineer of the Massachusetts Highway Commission, Boston, Mass. The Tyngsboro Road was laid in May, 1914; the Wayland and Natick roads in August, 1914.

The ordinary methods of heating and drying the sand were found satisfactory; the heaters, in fact, being home-made. Some of them were old corrugated metal culverts, 1½ ft. in diameter and about 12 ft. long; others were old, iron smoke stacks, about 18 ins. diameter and of varying lengths; still others were old boiler plates. Three or four of these heaters were used simultaneously, being laid parallel to each other. They were moved from time to time in order to have the hot sand near to the point of distribution, so that it could be moved in wheelbarrows from the heaters.

A small shallow trough was used for mixing the sand and tar. This trough was about 4 ft. long, 3 ft. wide and 1½ ft. deep. It has vertical ends, but otherwise was curved like half of a circular cylinder. In other words, it was approximately the shape of half of a barrel that had been split longitudinally. The sand used was very fine and clean, the fineness being required to provide for the suspension in the tar when pouring. The tar was brought to the road in tank wagons, fitted with steam coils. A steam road roller moved the tank wagons during distribution, and also provided steam for heating the coils.

The mixing trough, on a truck with small wheels, was attached immediately behind the tank wagon, and so close that the hot tar could flow from the tank wagon into the trough. The hot sand was brought up from the roadside and measured into the trough, and two laborers on each side mixed the tar and sand in approximately equal proportions, with the aid of pieces of hose. In the rear of the trough was a pipe outlet and a valve through which the mixed tar and sand were let into pouring pots. The usual flat-nozzled pouring pot was used.

Great care was taken in pouring, it being done immediately after mixing, and longitudinally with the road. It was found better to have only one man to do the actual pouring all the time, the pouring pots being carried to him from the mixing trough. This man acquired great skill in pouring and obtained uniform results in the quantity applied.

The broken stone was not less than 1½ ins. and up to about 2½ ins. to 3 ins., effort being made to secure stone as uniform as possible. The thickness after rolling was from 2 ins. to 3 ins., as desired. Before pouring, the stone was rolled sufficiently to prevent it from being rutted by wagon wheels. A fairly safe rule for the quantity of mixture was found to be, one gallon of mixture per inch of thickness in stone. Immediately after pouring, a thin covering of peastone was spread, sufficient in quantity to permit rolling without the roller sticking to the tar. After a thorough rolling, the surplus peastone was swept off, and an application of tar spread uniformly, preferably from a pressure distributor, and at the rate of about half a gallon per square yard. This, in turn, was covered with peastone or chips and thoroughly rolled.

The base, or bottom course of stone, was well drained, and before the top course was spread it was rolled thoroughly, and bound with stone screenings, fine screened gravel or sand. This was not only to preserve its shape during future operations, but also to prevent the escape of the tar-sand down into the bottom course.

A portion of the 300-ft. section that was laid with asphalt, was laid with material of about 140 penetration, the remainder being laid with asphalt of about 45 penetration. It was decided that the penetration should probably be about 100, although the stiffness of the asphalt might be increased according to the weight of traffic. This would also depend somewhat upon the type of asphalt used. The temperature to which the asphalt should be heated would vary according to the stiffness of the asphalt, but in any case should not be so high that it would be much hotter than the sand, as otherwise the mixture would likely foam. It was found that 300° F. was usually about the proper heat.

The Massachusetts engineers state that the cost of the bituminous sand-grout pavement should not be more than ten cents per square yard greater than the cost of the usual penetration type of road. The Massachusetts work, where costs were kept, was not over eight cents in excess of penetration work, and one job ran as low as six cents additional.

SHIPBUILDING ACTIVITY IN DENMARK.

The Copenhagen Floating Dock and Shipbuilding Company has secured a site of 70,000 sq. m. for extending the works; for this purpose the capital is being increased by 1,000,000 kronen, to 3,000,000 kronen. (1 krone = 27c.)

ROAD MATERIAL SURVEYS.

MEMOIR 85 of the Department of Mines, Ottawa, entitled "Road Material Surveys in 1914," by L. Reinecke, has just been published. The volume contains 244 pages, a considerable portion of which is consumed in compilation of general matter. These include the cost of various quarrying operations, transportation facilities from important rock deposits to sections where road metal is required, and local costs of various types of roads. The author's desire in furnishing this information is that it may be helpful to communities improving their road systems.

The preliminary discussion of various types of roads is somewhat elementary and may in sections merit friendly criticism. In the main, however, the data is good and furnishes valuable basic information.

The object of the surveys which have since been and are now being further extended, is to acquire a greater knowledge of our resources of good rock for road metal, thus rendering available more accurate data to assist highway departments in solving the difficult problem of supplying good roads of the highest economic quality at minimum cost.

The territorial surveys recorded are limited sections of the north shore of Lake Huron and adjacent islands, Essex County, Kent County and the north shore of Lake Ontario between Port Hope and Hamilton.

Previous geological surveys showed that large areas of diabase rock existed along the north shore of Lake Huron. In fact, for some years past quarries have been operated in this section. A large amount of rock is crushed, the greater portion of which is exported. The rock has been known to the trade as "trap." This term is somewhat general. According to this report, it includes the "fine-grained, dark, volcanic rocks, the andesites, augite andesites, basalts and other more basic black volcanic rocks, and the porphyritic equivalents of these rocks, including diabase."

The Lake Huron diabase deposits were found to be of high quality. Samples taken from various sections generally showed a toughness of 18 or over. Those taken from the quarry at Bruce Mines ran as high as 27. The major "per cent. of wear" test ran from 2 to 4.37. The major portion of samples stood close to 2.5. The rock showed from 40 to 100 in cementing value. Such tests would indicate, as past experience has also proven, that much of this rock is suitable to withstand "heavy traffic" as this term is applied to macadam roads. It is also of excellent quality to be used in asphaltic concrete pavements.

The extensiveness of the deposits permits production on a large scale at relatively low cost. It must be understood, of course, that the very qualities of this rock which make it so valuable for road work cause it to be difficult and expensive to quarry and crush. Veins of epidote and quartz, moreover, render drilling slow, and large fragments of rock constantly fail to shatter until additional small shots are used. The quarrying expenses will, accordingly, always prove high as compared with those of the softer and poorer types of rock.

The location of this material is somewhat distant from central Ontario. However, deep channels facilitate cheap water transportation to the various lower Ontario points of discharge.

Essex County contains little road material of high quality. Gravel deposits, however, are abundantly distributed throughout various sections and are of no mean economic importance in the development of better roads. Field stone, moreover, could be made to yield considerable material. In addition there are supplies of gravel in adjacent territory and also limestone deposits on Pelee Island.

In Kent County, again, the gravels, many of which are of good cementing quality, form the bulk of the local road material.

A territory of 130 miles in length and from one to seven miles in width was investigated along the north shore of Lake Ontario from Port Hope westward. The gravel deposits proved extensive. They carried a uniform average of 70 per cent. limestone, 20 to 30 per cent. hard pebbles, and 0 to 10 per cent. soft pebbles. The cementing value of this material is high. Local roads constructed of these gravels unprotected by a bituminous covering stand up under medium traffic from 2 to 3 years. They do not prove satisfactory, however, with simply

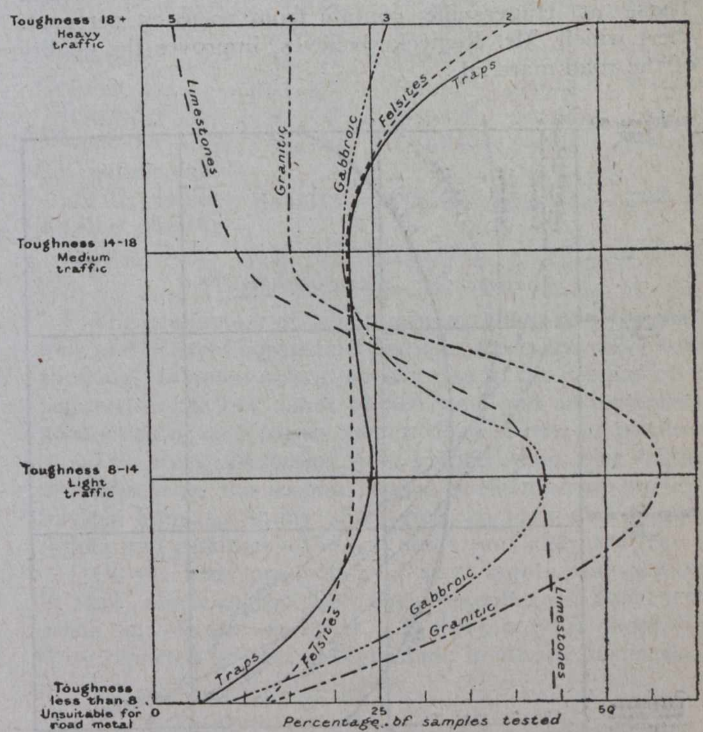


Fig. 1.

Curves showing the relative toughness of various groups of rocks and their consequent behavior under traffic (based upon tests made in the laboratory of the Office of Public Roads, U.S. Department of Agriculture, Washington, D.C.).

Rock grouping used in the diagrams.

1. Traps, including the finer-grained, dark-colored igneous rocks; andesite, augite andesite, basalt, diabase, basaltic and andesitic tuff, breccia, etc.
2. Felsites, including the finer-grained, light-colored igneous rocks; rhyolite, quartz-porphyry, trachyte, etc.
3. Gabbroic rocks, including the coarser-grained, dark-colored igneous rocks: diorite, augite-diorite, anorthosite, gabbro, peridotite, pyroxenite, etc.
4. Granitic rocks, including the coarser-grained, light-colored igneous rocks, granite, syenite, monzonite, granodiorite, quartz-diorite, etc.
5. Limestones, including all varieties of limestone and dolomite.

water consolidation when the traffic is fairly heavy. When Portland cement is to be used as a binder, it is found advisable in many cases to screen and mix, as the aggregate varies greatly even in limited sections.

Large boulders of glacial origin are found on the terrace between the old Lake Iroquois beach line and the present shores of Lake Ontario. These deposits are not in close enough proximity to existing and projected roads to be of great economic value at the present time.

The outside sources of supply represent excellent factors as a solution in building projected roads. The trap deposit on the north shore of Lake Huron and at Havelock, as well as the limestone largely quarried at Hagersville, Dundas and other adjacent sections, and the granite at Gananoque, merit consideration in this connection. The limestone deposits referred to are well-known. Those at Hagersville contain from 10 to 25 per cent. chert which, Mr. Reinecke believes, improves the quality of the road material.

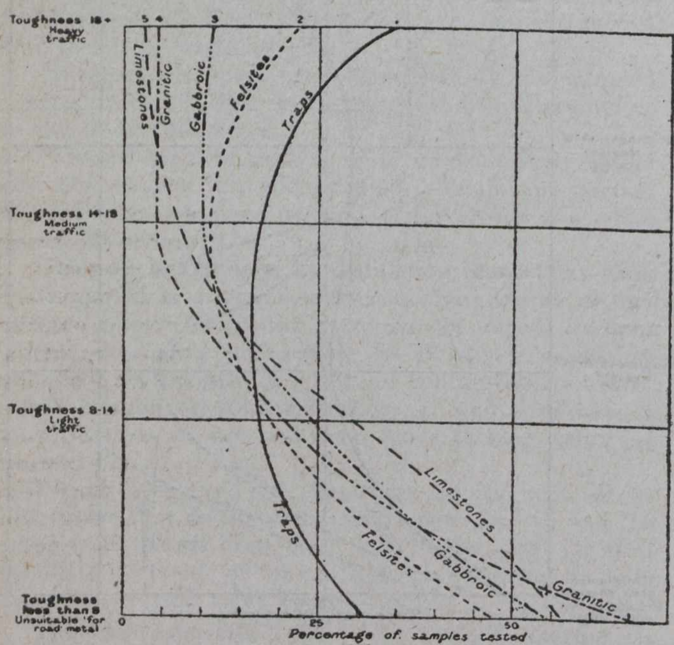


Fig. 2.

Curves showing the relative toughness and consequent behavior under traffic of such rocks as have a cementing value high enough to permit of their use on macadam roads (based upon tests made in the laboratory of the Office of Public Roads, U.S. Department of Agriculture, Washington, D.C.

The accompanying diagrams, together with their respective interpretations, will prove interesting. It is readily seen that trap as a whole furnishes the best road metal. In general, the remaining order of merit is as follows: Felsites, gabbroic, granite, and limestone.

From a careful study of curves such as these, the important part which the quality of metal plays in the success with which a road will withstand traffic can be readily seen. It further serves to illustrate a very important truth, namely, that in selecting the most economical type of road to withstand a given amount of traffic a cheap type such as water-bound macadam should not always be overlooked. While this type of road may fail and consequently prove unsatisfactory if constructed with limestone or granite, were it built with good felsite or trap, its merits might be unquestioned. If the yearly charges against a water-bound macadam road, to provide adequate maintenance and interest and sinking fund charges, do not exceed the combined amount of these charges against a more expensive road, the water-bound macadam should be selected, assuming that it will give as efficient service while in use as will the more expensive road.

It is encouraging to know that our government is doing its share in furnishing data to aid materially in the scientific solution of road problems.

ENGINEERS WILL OFFER SERVICES.

Wills Maclachlan, secretary of the Canadian Electrical Association, was in Ottawa recently, and in conversation with F. T. C. O'Hara, Deputy Minister of Trade and Commerce, Mr. Maclachlan said that engineers who could not enlist are desirous of doing something to aid the government in prosecuting the war. Mr. O'Hara intimated that the government does not understand clearly just how engineers might like to help, and he suggested to Mr. Maclachlan that the engineers should get together and present some definite plan of action to the government.

Upon his return home, Wills Maclachlan wrote to the executives and officials of various engineering societies centred at Toronto and invited them to meet at the Engineers' Club last Tuesday evening. The following gentlemen accepted the invitation: Chester B. Hamilton, R. B. Wolsey, W. A. Bucke, B. G. Buchanan, W. H. Thom, Ernest V. Pannell, R. K. Shepard, Childes C. Clark, S. L. B. Lines, Walter Carr, T. W. Gibson, J. F. Neild, L. N. Arkley, S. B. Chadsey, J. C. Armer, A. H. Hull, W. E. Segsworth, E. P. Mathewson, E. M. Ashworth, E. J. T. Brandon, Alfred Burton and Wills Maclachlan.

The above gentlemen belong to one or another of various societies, including the American Society of Mechanical Engineers, the Engineers' Club of Toronto, the Society of Chemical Industry, the Toronto Section of the American Institute of Electrical Engineers, the Toronto Chapter of the Ontario Association of Architects, the Institute of Electrical Engineers of England, the Canadian Mining Institute and the Canadian Society of Civil Engineers. While they were not officially representing any of the above societies at the meeting last Thursday evening, it is thought that they will no doubt be able to interest officially the various societies to which they belong, and to secure their co-operation in the movement if they can evolve any practical plans for assistance by engineers other than enlisting or volunteering for munition work under the direction of the National Service Board.

Discussion at the meeting last Thursday evening brought out a wide range of suggestions and a committee was appointed to consider them. Canadian engineers and scientists are invited by Wills Maclachlan to send him any suggestions they would like to make, regarding ways in which engineers can be of use at home in furthering the war.

SHIPBUILDING IN SWEDEN.

The Finnboda yard, near Stockholm, has decided to construct a new slip, capable of accommodating vessels 375 ft. long and of 5,000 to 6,000 tons. The necessary blasting operations will be commenced shortly, and the work is expected to be so expedited that the first keel can be laid in the summer of 1917. The first boat to be built will, it is stated, be only 250 ft. long.

Editorial

REPORTS FROM MUNICIPAL ENGINEERS.

Many municipal engineers will during the next few weeks be busy preparing their reports of work done during the year which has just closed. It is to be hoped that the practice of preparing written reports whenever possible will be more general in the future than has been the case in the past. There has been a tendency on the part of too many municipal engineers to evade written reports.

The value to one municipality of how work is progressing in others, what methods are being employed to advantage is obvious. To the municipal engineer falls the duty of supplying his corporation with exact and complete data, in order that such data may be of use not only to himself and the community which he serves at the time, but also to other municipalities.

It is, however, to himself that the preparation and presentation of such reports will be most valuable. Access to such records as these will have a direct bearing on his future success, if only for the purpose of having an exact record of all his reports of progress, proposals, opinions, etc., as furnished to works committees and council, so that when an emergency arises it will be possible for him to make immediate and exact repetitions of statements made perhaps months before. In this way the engineer who has fortified himself against the freaks of memory will find in it a safeguard to his professional standing.

The municipal engineer of to-day is called upon to express opinions upon so many different phases of civil engineering work that it is incumbent upon him, perhaps more than any other class, to take this exhortation (not to trust too much to memory but make written reports) especially to heart.

The municipal engineer must have a very diversified knowledge of engineering. He is called upon to deal with problems in which a knowledge of hydraulics, sanitation, surveying, railways, roads and pavements, etc., etc., are called for, all of which makes it important not to be hampered by a failure of memory.

Finally, a report emanating from a municipal engineer should be complete in itself, making oral explanations unnecessary.

IRON AND STEEL INDUSTRY DURING 1916.

The growth of Canada's iron and steel industry has of late years been phenomenal. While in 1914 the industry was at a low ebb, it is gratifying to note that to-day Canada ranks eighth among the steel-producing countries of the world.

In the report on the production of iron and steel during 1916, just made by Mr. John McLeish, B.A.F.S.S., who is in charge of the division of mineral resources and statistics, the following highly interesting facts are given covering the activity in this field of effort during 1916:—

The records received from the producers show that the production of pig iron and of steel ingots and castings during the first eleven months of the year which together with estimates for December show a probable production of pig iron in Canada during the twelve months ending December 31, 1916, of 1,171,727 short tons (1,046,185 gross tons) and a probable production of steel ingots and direct steel castings of 1,454,124 short tons (1,298,325 gross tons), of which 1,423,485 short tons were steel ingots and 30,639 short tons were direct castings.

The production in 1916 during the first six months and monthly during the last six months was as follows, in gross tons:—

	Pig iron, gross tons.	Steel ingots, gross tons.	Direct castings, gross tons.	Total, gross tons.
Six months ending June ...	501,872	577,999	11,715	589,714
July	82,154	101,178	2,284	103,462
August	78,450	108,889	2,299	111,188
September ...	91,736	116,828	2,524	119,352
October	101,436	126,577	2,924	129,601
*November ..	95,237	119,468	2,745	122,213
†December ...	95,300	119,930	2,865	122,795
Six months ending December	544,313	692,970	15,641	708,611
Twelve months ending Dec..	1,046,185	1,270,969	27,356	1,298,325

*Partly estimated. †Estimated.

The production of pig iron in 1915 was 913,775 short tons and of steel ingots and castings 1,020,896 short tons, showing, as noted above, an increase in the production of pig iron in 1916 of about 28 per cent. and an increase in production of steel ingots and castings of over 42 per cent.

The 1916 production was greater than that of any previous year, the second largest production of pig iron having been 1,128,967 short tons in 1913 and of steel ingots and castings 1,168,993 short tons also in 1913.

Of the total production of steel ingots and castings in 1916, about 43,790 short tons (39,098 gross tons) were made in electric furnaces. In 1915 only 61 short tons were reported as having been made in electric furnaces.

BITUMINOUS SAND-GROUT PAVEMENT.

Under the above head an article appears on page 17 of this issue, descriptive of bituminous sand-grout pavements laid in Liverpool, Eng., and near Boston, Mass.

While the Boston roads are purely experimental and are not old enough to be particularly significant, it is understood that some of the Liverpool roads have been down as long as fourteen years, and are still in good condition. So far as *The Canadian Engineer* is aware, no roads of this type have been built in Canada, but the success of these roads in England will undoubtedly lead to their trial in this country.

The suggestion is made, therefore, that it would be of distinct interest to highway engineers and road contractors in Canada, if the Canadian & International Good Roads Congress, which is to meet this winter in Winnipeg, could induce Mr. J. A. Brodie, city engineer of Liverpool, to attend the convention and deliver a thorough paper upon this subject. Mr. Brodie is generally admitted to be one of the leading road authorities in England, and his visit to Canada at this time would be most opportune in view of the large number of roads that will be built after the war is over.

The city engineers throughout Canada would warmly welcome a visit from Mr. Brodie, and the members of the Canadian Society of Civil Engineers would, no doubt, co-operate in his entertainment.

PERSONAL

H. McEWEN, superintendent of the Prince Edward Island Railway, Charlottetown, P.E.I., has retired after 42 years' service.

E. W. DELANO, formerly of the engineering department, Bangor & Aroostook Railway, has been appointed division engineer, Lake Superior District, C.N.R., with office at Capreol, Ont.

H G. BARBER, formerly resident engineer, Canadian Pacific Railway, Nelson, B.C., is now a lieutenant in the 239th Battalion Overseas Railway Construction Corps, Canadian Expeditionary Force.

A. C. BEDFORD has been elected president of the Standard Oil Company of New Jersey, to succeed the late John D. Archbold. Mr. Bedford has for some years been vice-president and treasurer of the company.

JOHN G. SULLIVAN, M.Can.Soc.C.E., chief engineer of the western lines of the Canadian Pacific Railway, has been announced a nominee for president of the American Railway Engineering Association for the coming year.

G. H. TRIPPLEHORN has resigned his position with the Chatham Cement Tile Company, with which he has been associated for a number of years, and has gone to Detroit, where he will assume the management of a cement company.

H. G. SCHANCHE, who has been with the Laurentide Company, Limited, Montreal, and is now finishing his forestry work at Pennsylvania State College, has been elected an associate member of the Canadian Society of Forest Engineers.

J. R. W. AMBROSE, M.Can.Soc.C.E., chief engineer of Toronto Terminals Railway Co., Toronto, has been named as one of the ten candidates from which the nominating committee of the American Railway Engineering Association is to be selected.

J. A. MacDONELL, formerly head of the contracting firm of MacDonell, Gzowski & Company, Vancouver, who went overseas with the First Canadian Pioneers, has been promoted to the command of that corps. He has also been recommended for the D.S.O.

R. COLCLOUGH, formerly superintendent, District 1, International Division, Canadian Government Railways, Levis, Que., has been appointed superintendent of District 1, Transcontinental Division, Quebec, Que., succeeding J. E. Morazain, transferred.

Lieut. K. L. DUGGAN, of the Mounted Rifles, son of Mr. G. H. Duggan, of Montreal, president of the Canadian Society of Civil Engineers, has been reported wounded in action. Mr. Duggan's other son, Lieut. H. S. Duggan, was killed in action last spring.

W. H. FORTIER, of the Dominion Glazed Cement Pipe Company, and Sanitary Engineer DALZELL, of Vancouver, will, at the request of the Vancouver Board of Works, make tests regarding the relative merits of cement pipe and vitrified pipe, and submit a report.

W. A. COWAN, A.M.Can.Soc.C.E., formerly division engineer, Transcontinental Division (Ontario), Canadian Government Railways, has been appointed acting general superintendent, Transcontinental Division, Cochrane, Ont., during the absence of F. P. Brady, on account of ill-health.

Dr. H. N. WHITFORD and ROLAND D. CRAIG, having completed their report to the Commission of Conservation, Ottawa, concerning the area of merchantable timber in British Columbia, have now left that province, the former to take up a position in the Forest School, Yale University, and the latter having gone to Ottawa.

F. A. CHISHOLM, A.M.Can.Soc.C.E., formerly superintendent of the St. John's, P.Q., division of the Southern Canada Power Company, has been appointed superintendent of power of the Sherbrooke Railway and Power Company, vice J. T. Kemp, resigned. A. P. BROADHEAD, superintendent of the Drummondville division of the Southern Canada Power Company, succeeds Mr. Chisholm at St. John's, while H. P. FISK takes the place of Mr. Broadhead at Drummondville.

J. H. CUNNINGHAM, of Ladysmith, Vancouver Island, B.C., who only recently returned from a visit to Nova Scotia, has resigned as manager of the Extension colliery of the Canadian Collieries (Dunsmuir), Limited, and has been appointed superintendent of the coal mines of the Nova Scotia Steel and Coal Co. He left Ladysmith for the east early in December. He has been succeeded at Extension by T. A. Spruston, who has been manager of the same company's No. 7 mine, Comox colliery, VI.

CLYDE LEAVITT, of the Forestry Branch, Ottawa, will attend an international forestry conference in Washington, January 18th and 19th, at which special efforts will be made to obtain national and international co-operation in a campaign to check the spread of the white pine blister disease. The conference has been convened by P. S. Ridsdale, secretary of the American Forestry Association, Washington.

CANADIAN SOCIETY OF CIVIL ENGINEERS, CALGARY BRANCH.

One of the most interesting and instructive war addresses delivered in Calgary was given Tuesday evening, December 26th, at a smoker held by the Calgary Branch of the Canadian Society of Civil Engineers. The speaker was Capt. H. Sidenius, headquarters staff officer of military district No. 13. "Military Engineering and Trench Warfare in the European War" was his subject. This he handled capably and illustrated his talk by the use of drawings on a blackboard as he proceeded.

Capt. Sidenius went thoroughly into the technical details of the subject, explaining by means of diagrams and maps the methods of building the various kinds of trenches necessary in the present war. He also described the construction of obstacles and wire entanglements, machine gun emplacements, telephone connections between the front trenches and the several official stations behind the lines, as well as the details of organization of trench defence and attacks. He exhibited maps prepared for air craft observations at Ypres last spring. A feature which illustrates the marvelous efficiency of the military organization at the front was brought out in his explanation that the results of air craft observations made in the forenoon was issued in the afternoon of the same day on completely finished maps showing the exact location of the enemy trenches as well as all other natural or artificial features of the ground.

Capt. Sidenius gave a general explanation of the organization of trench warfare, and at the conclusion was given hearty applause and a vote of thanks by the society. Mr. A. S. Dawson presided.

More than 30 per cent. of the members of the Calgary branch have already gone to the front, and it is the intention of the Society to unveil a roll of honor at an early date.