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Contents of this issue on page 599

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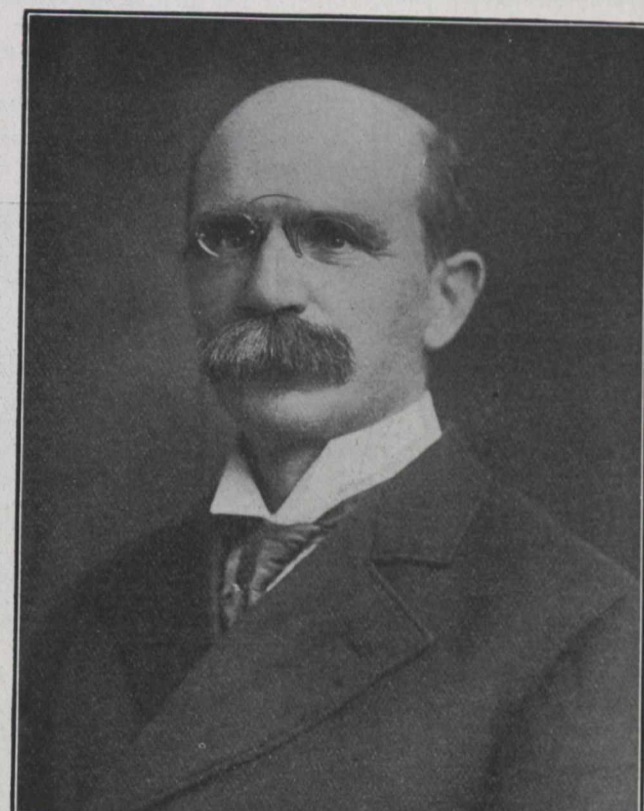
MUNICIPAL ENGINEERING, CITY OF TORONTO

The past year has seen great municipal changes take place in the city of Toronto, and outlying districts. These changes and reorganizations have necessitated technical attention in nearly every case, as may be gathered by even a short perusal of the city engineer's report for 1910. The principal construction works may be included in a list composed of the filtration plant, sewage disposal work, municipal railway work, grade reorganization of Grand Trunk Railway, bridge construction. The city engineer designing and superintending these various works is Mr. C. H. Rust, who is this year president of the Canadian Society of Civil Engineers.



Mr. C. H. Rust, City Engineer of Toronto, President of Canadian Society of Civil Engineers.

Figure 1 will, in a small measure, present some tangible idea of the increased cost of construction and maintenance for the engineering departments from the year 1897 to 1910.

The filtration plant, now almost completed, has been constructed on the island opposite to John street. The plant is designed as a battery of twelve filters built almost entirely of concrete. Fig. 2 is an illustration of the interior of a single unit in this purifying work. The filtration plant and sewage disposal works may be considered as joint undertakings, both being necessary owing to the former method whereby the discharge entered the lake in dangerous proximity to the waterworks intake. The main currents of Lake Ontario naturally assume an eastern course, and this has influenced the location of the disposal works at point considerably to the east of the intake and filtration plant. While this is the main object of the disposal works, the design also allows the discharge to enter the lake in such a condition, chemically and bacteriologically, as to be harmless to other municipalities drawing their supply from Lake Ontario further down the shore. The trunk sewers and sewage disposal works are estimated to cost $2,400,000. A great deal of the sewer construction has been done by tunnelling to avoid street blocking. A difficult portion of the work was encountered in carrying the main sewer under the Don River, where cast iron pipes had to be laid to act as syphons.

This syphon consists of two lines of cast iron pipe, and it was decided that it should be laid in the dry so that the joints could be properly made with lead and so that better inspection could be had; to this end, the contractor built coffer dams across the Don River, both above and below the line of the syphon, to prevent the water from the river and from the bay entering into the excavation. The material was of a fairly treacherous nature, and required extremely heavy sheeting. Work was commenced on this contract in July, and at the present time laying of the pipes has been successfully completed from one end of the syphon to the other, and the coffer dams partly removed. In order to verify our opinions with regard to the strength of the cast iron pipe used on the work, a full sized test was carried out.

The total amount of high level interceptor under construction is 29,880 lineal feet. This is the length from Garrison Creek sewer to the Disposal Works. There is also to be constructed under the by-law an extension of the high level interceptor from this point westerly to the west end of Springhurst Avenue, where the avenue turns to the north.

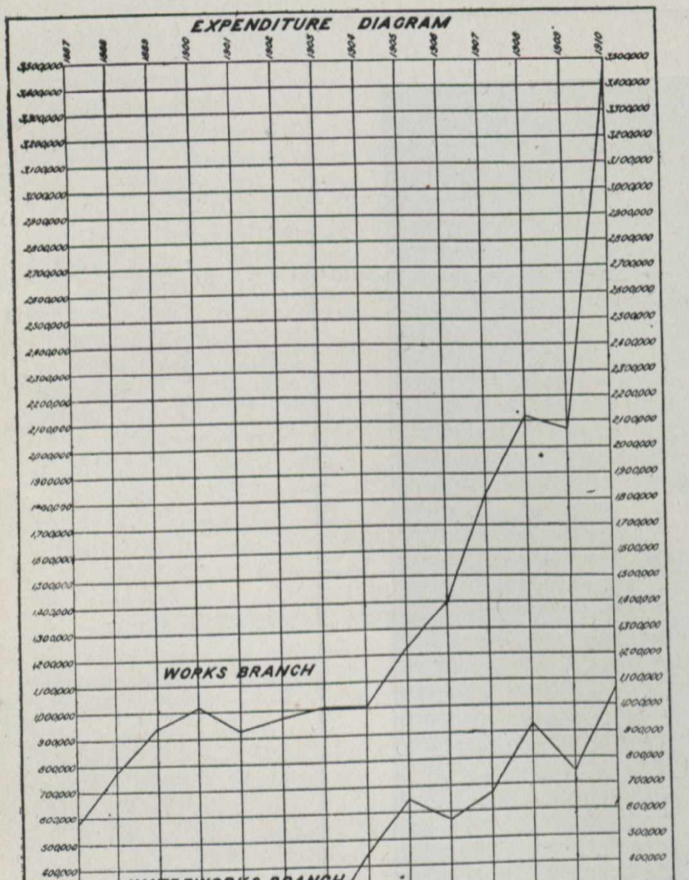


Fig. 1.

In order to make the system complete, the engineer in charge has recommended that authority be granted to carry this sewer from this point westerly to Roncesvalles Avenue and the corner of Queen and King Streets, so that the sewage from West Toronto coming down the proposed new trunk sewer from Bloor Street to this intersection could be intercepted, and the dry weather flow carried easterly to the Disposal Works.

Fig. 3 is a plan showing the lines laid down for this sewage system, the point marked X is about the spot of the waterworks intake. Figs. 4 and 5 are illustrative of the method, mentioned above, to carry the conduits across or under the Don River.

Railway engineering in the vicinity of Toronto has been very active the past year or more, as a result of the order of the Board of Railway Commissioners to eliminate level crossings at the Exhibition entrance, Parkdale, Sunnyside, and along the Lake Shore towards the western portion of the city. This, of course, may be taken as the larger portion of railway work; however, considerable other railway construction has been undertaken. The grade revision work is being undertaken jointly by the Grand Trunk Railway and the City of Toronto, and will be, when completed, the most extensive grade separation work ever undertaken in the Dominion of Canada.

The work consists of grade separation, starting from C.P.R. diamond just west of Strachan Avenue, and continuing in a westerly direction to Mimico, a distance of about 5.95 miles.

The first grade crossing to be separated is Dufferin Street, followed by Dunn, Jameson, Dowling, Queen Street (Sunnyside), Indian Road, Howard Avenue, Ellis Avenue, Windermere Avenue, Jane Street, Queen Street West, Trafalgar and Church Street, Mimico, comprising in all grade separation for thirteen crossings. The last crossing in the present city limits of the above mentioned highways is Howard Park Avenue, or what is sometimes known as western entrance to High Park.

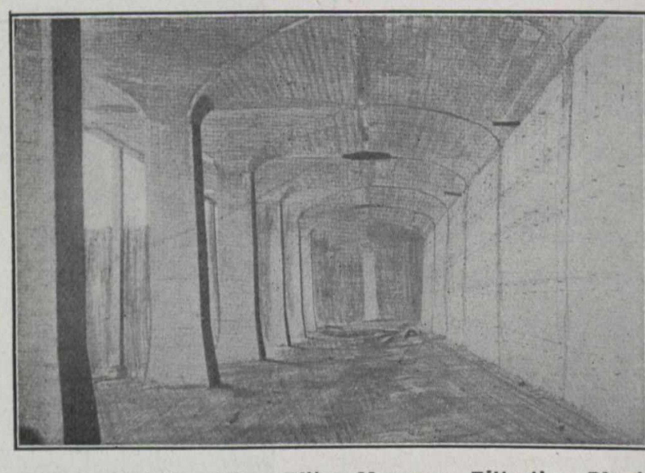


Fig. 2.—Interior View Filter Masonry, Filtration Plant.

The tracks will be depressed at Dufferin Street 23 ft., Dunn Avenue 25 ft., Jameson Avenue 25 ft., Dowling Avenue 25 ft., at Queen Street, Sunnyside, the tracks will be elevated 2.5 ft., Indian Road 12 ft., Howard Park Avenue 15.5 ft., Ellis Avenue 16 ft., Windemere Avenue 14.5 ft., Jane Street 12.5 ft., Humber River bridge 12.5 ft., Queen Street 12 ft., Salisbury Avenue, Church Street; that is, the present grade crossings at Dufferin, Dunn, Jameson and Dowling Avenues will be overhead bridges upon the completion of the work, and the remaining crossings, from that point westerly, will all be subways. The maximum grade of the railway tracks when the work is completed will be four-tenths of one per cent. (0.40 per cent.) in either direction.

The cost of this work is estimated at $1,800,000.00. The accompanying diagrams will make the general purpose of this work more clear. Figure 6 is a map of the work.

Since 1905, applications have been made from time to time for the construction of railway tracks on the reserved allowance under the Don Improvement plan on the east side of the River Don.

In view of the fact that the city did not desire to give the railway rights at this point to any one railroad, and in addition consideration of the demand for railroad accommodation in the manner of team tracks at this point, it was decided that the city should construct and own these tracks, granting operating rights to all railroads on a rental basis to such time as the viaduct question was definitely settled,

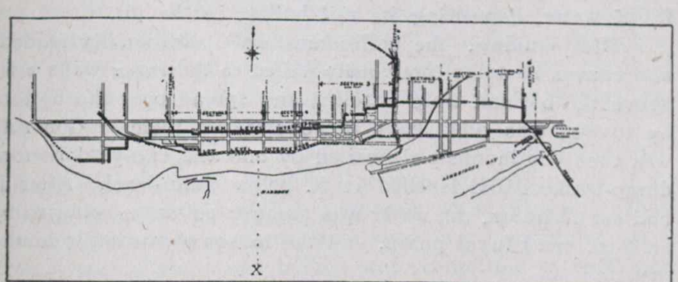


Fig. 3.

after which these tracks are to form part of the General Ashbridge Bay plans for railway facilities, which will be owned and operated by the city.

The trackage to be constructed consists of 1.96 miles. It is single track from the C.P.R. connection near subway at Winchester Street through to Gerrard Street; from this point it is double track to its present southern terminus, which is the Grand Trunk right-of-way south of Eastern Avenue.

In the construction of these tracks it was necessary to remove about 30,000 cubic yards of gumbo clay located in the Isolation Hospital hill, immediately north of Gerrard Street bridge. A point probably worthy of mention is the fact that practically all of the 30,000 yards had to be blasted. The fact that the Isolation Hospital was only about 200 feet from the cut naturally caused some hesitancy on the part of the Department. It was all blasted without mishap or complaint.

Summary of cost:—

Total original estimate	$37,000.00
Cost of work to date	29,035.54
Credit balance	7,964.46
Estimated cost for completion	3,964.46
Balance	$ 4,000.00

Fig. 4.—Cast Iron Pipe, Don Syphon High Level Interceptor.

A point which might be mentioned for the benefit of anyone having to handle similar material, is the fact that 8 sticks of dynamite (50 per cent. nitro-glycerine) had no effect on this material other than to blow a hole about 3 feet in diameter, and black blasting powder alone was practically useless. After 3 days of experimenting, it was found that 3 sticks of dynamite and 2 quarts of blasting powder in each hole, 5 to 10 holes at a time, drilled alternately 5 feet apart, and alternately 6 feet and 2 feet back from the face, had the desired effect; it loosened from 21 to 40 yards at a time, depending on the number of holes drilled. The average haul was 1,600 feet on dump cars drawn by teams; team rate $5 per day for nine hours; labor $2 per day for nine hours. The average cost was 29.5 cents per cubic yard.

Bridge work during the past year has been particularly active owing to three large works being placed underway, viz., Wilton Avenue and Queen Street bridges and elaborate alterations to the Dundas Street bridge crossing the steam railway tracks.

In a former article, appearing in this paper, there was presented a description of the Queen Street viaduct.

The design calls for a 129-ft. 6-in. through truss span crossing the river supported on 10-ft. towers carried by the old masonry. The east approach commences at a point 34 ft. east of the centre line of Carroll Street, and is filled to a point 58 ft. 6 in. west of the centre line of Davies Avenue.

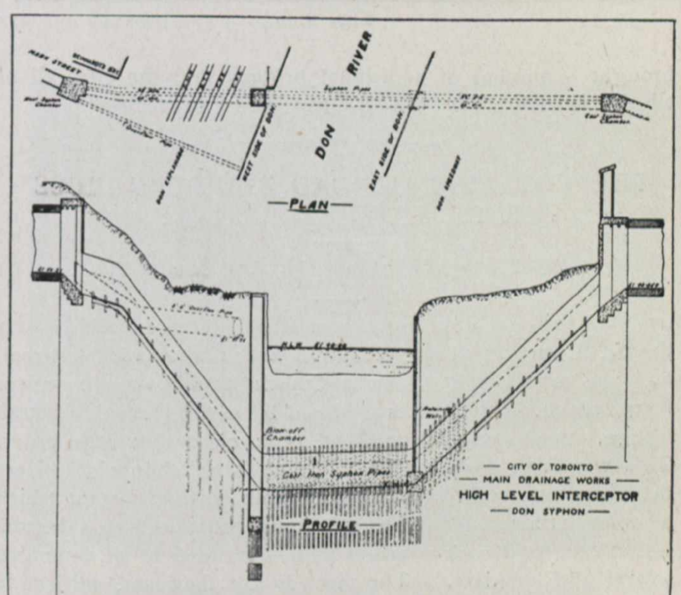


Fig. 5.

From this point there are 48-ft. and 65-ft. plate girder spans crossing over the Don Roadway and the reservation for railway tracks respectively, and connecting with the tower supporting the river span.

The maximum grade on the Queen Street approaches is 3 per cent., and that on the King Street approach 4 per cent., as compared with 3.3 per cent. and 1.58 per cent. respectively on the former roadways.

Access to the roadways on each side of the river is given on the west by a new roadway parallel to and immediately south of Queen Street leading from the Don Esplanade to River Street, and on the east by ramps to the Don Roadway, both north and south of Queen Street.

Including the approaches, the bridge has a total length of 1,300 feet, of which 579½ feet is carried on steel viaduct. The total weight of steel is estimated as 984½ tons. Throughout the entire length the roadway has a total width of 42 feet between curbs, and is paved with wood blocks on concrete.

The substructure is of concrete, that portion east of the river being carried on spread footings on hard blue clay. That part west of the river on wooden piles cut off below low water level, except the abutments and retaining walls, which are carried on spread footings.

The bridge is designed for 40-ton cars in accordance with the specifications of the Ontario Railway and Municipal Board.

A total of $10,986.86 has been spent on bridge maintenance during the past year, as compared with $9,947.22 in 1909, and $12,347.25 in 1908. This is very satisfactory, especially when it is considered that recent annexations have

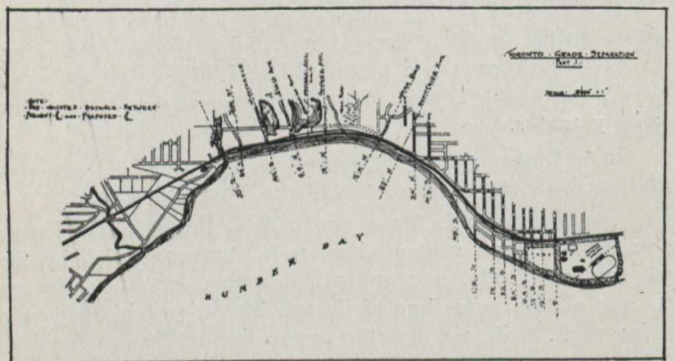


Fig. 6.

brought a number of additional bridges into the city, all of which have been in a deplorable state of repair.

THE COTEAU RAILROAD BRIDGE ACROSS THE ST. LAWRENCE RIVER.*

By Frank W. Skinner, M. Am. Soc. C.E.

The original crossing of the Grand Trunk Railway over the St. Lawrence River, about 37 miles west of Montreal, and one mile above the swift Coteau Rapids, was by a car ferry, which, in 1888, was replaced by a single-track through bridge, about 4,025 ft. long and 25 ft. clear above the water. This was built over two islands and three channels. Here the water was 23 ft. in maximum depth, with a current which at times attained 8½ miles per hour, moving over a bottom of smooth hard rock overlaid with 3 ft. to 6 ft. of cemented gravel and boulders. The river is not, however, subject to freshets, and for about seven months in the year is navigable at this point for large boats going down stream. In the spring the channel is filled with enormous quantities of heavy ice.

There were four 223-ft. spans over the south channel, ten 217-ft. spans over the middle channel, two 175-ft. and one 139-ft. fixed spans, and one 355-ft. swing span over the steam-boat channel, which, as the plan shows, is adjacent to the north shore. The superstructure, weighing about 2,750 tons, had double intersection trusses with riveted connections, and was supported on 8-ft. by 24-ft. masonry piers on concrete footings, built in open wooden cofferdams, and located by measurements made on the ice.

The cofferdams for the fifteen fixed river piers were 20 ft. wide and 67 ft. long, with pointed ends and braced walls of 12-in. by 12-in. timber. They were built about a mile up-stream from the bridge site, and floated to a position suspended from a pair of 24-in. by 24-in. oak beams on 20-ft. towers on the decks of a pair of barges towed by from two to five tugs. When within 800 ft. of the site, they were secured by five heavy anchors, the tugs released, and the anchor cables slacked off until the cofferdam was about 25 ft. up-stream of the required position, when it was heavily loaded with steel rails and lowered nearly to the bottom by the four nine-part tackles previously used to lift it while passing through shoal water. Both anchor and suspension tackles were then simultaneously slacked to land it in exact position. The cofferdams were sunk in pits excavated to rock in about 15 days each by two dipper dredges. The swift current frequently broke the 18-in. by 20-in. oak anchor spuds of the dredges, and on one occasion overturned a cofferdam in 30 ft. of water, depositing its rail ballast in the pit.

After landing, the cofferdams were additionally loaded, and canvas curtains, previously nailed to the inner walls 2 ft. above the bottom, were unrolled and spread over the bottom by divers, who piled bags of concrete on them. Concrete was then continuously deposited by one and two-yard bottom dump-buckets and levelled 12 ft. below water-level. After it had set 48 hours, the water was pumped out in 30 minutes by an 8-in. centrifugal pump, and the masonry was built in the dry.

The fixed spans, weighing about 156 tons each, were erected on falsework on an island a mile above the bridge site, skidded on to barges, towed between the piers, and deposited on them by admission of water ballast to the barges. The bridge was put in service 10 months and 20 days after work on the substructure was commenced.

In order to provide for increased traffic and heavier loads the old superstructure was replaced in 1910 by a much heavier new one, on the same substructure, without seriously interrupting traffic. Seven of the channel spans are new, the three remaining ones being old trusses still in place. The new spans, like the old ones, were made and erected by the Dominion Bridge Company, Montreal, under the direction of Mr. Phelps Johnson, manager. The structure is riveted throughout. Its details were proportioned to take a load of two ten-wheel engines and tenders, followed by a uniform load of 5,000 lb. per foot run, but the floor-beams and stringers were also made strong enough to carry two concentrated loads of 62,000 lb. each, spaced at 7-ft. centres. The floor is of wooden ties. Guard-timbers, measuring 8 in. by 10 in., are provided. The most interestng feature of the bridge is, however, the method used to erect it and transfer it to place. A low pile falsework, 550 ft. long and 20 ft. high, was built

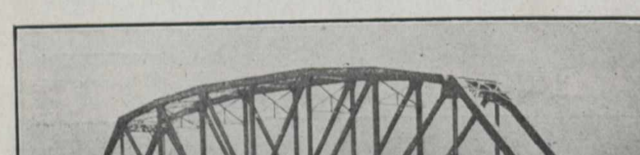


Fig. 1.

on the shore of one of the islands crossed by the bridge, and the steel, delivered from the shops alongside, was unloaded from the cars and erected on it by a travelling stiff-leg derrick moving from end to end of the falsework on a 32½-ft. gauge track on an independent falsework trestle.

The assembled spans, weighing 325 tons each, were supported at the second panel points from the ends on rollers moving on pile trestle piers about 200 ft. long, perpendicular to the erection falsework. The piers, 162 ft. apart, carried roller-tracks made of three lines of 15-in. I-beams graded down 2 per cent. towards the river end. The field-rivets

*Abstracted from an article in London "Engineering."

were driven while the next span was being erected in the same manner, after which it was moved out and riveted, and so on until seven spans were on the piers and an eighth on the erection falsework.

To effect the transfer two 38-ft. by 100-ft. steel-frame barges braced together, 81 ft. apart on centres in the case of the main span, and less in that of the shore, were partly filled with water ballast, and placed under the outermost span on the piers. The water ballast was pumped out, allowing the barges to rise until the span took bearing, at the second and third panel points from each end of each truss, on wooden towers, 14 ft. high, on the decks of the barges, and were lifted clear of the piers.

The barges, with a draught of 3½ ft. and a freeboard of 4½ ft., were then towed to a point a little up-stream of the piers, and secured to an anchored scow, Fig. 1. Their tackles were then slacked, and they were carefully eased down-stream, and water ballast was readmitted to sink them until they deposited the spans on the piers, after which they were towed back to the erecting-yard for another span, and so on for eleven of the fixed spans, the remaining six being erected in the usual manner, on ordinary falsework in shallow water at the bridge site.

While each new span was being lifted from the erection piers, floated down-stream and placed in permanent position on the piers, the corresponding old span was conversely lifted from the same piers by a pair of smaller wooden barges, operated by water ballast, towed up-stream, and deposited on the erection piers at the opposite end of the island falsework. It was then rolled down a 2 per cent. grade, to the shore ends of the falsework piers, and there taken apart by a wrecking-car, and the next span brought in, and so on. The movement of the spans was so arranged as to occur during the longest intervals between regular trains, and as it occupied but a few hours for each exchange of an old for a new span, little interference with the train service was occasioned.

Solid-steel live rollers, 2½ in. in diameter, were put under the spans on the falsework piers, and the spans were moved by two four-part tackles operated by the hoisting engine on the erecting traveller. The new-span barges were stiffened by five longitudinal steel trusses; they drew 19 in. light, and were equipped with one 8-in. centrifugal pump on each, to handle the water ballast in and out. The old span barges had a draught of 15 in. light and 21 in. loaded, and were each provided with a 6-in. gate and a 6-in. centrifugal pump supplied with steam from a boiler on deck.

The new bridge was designed and erected under the direction of the engineering department of the Grand Trunk Railway System, Mr. Howard G. Kelley being the chief engineer.

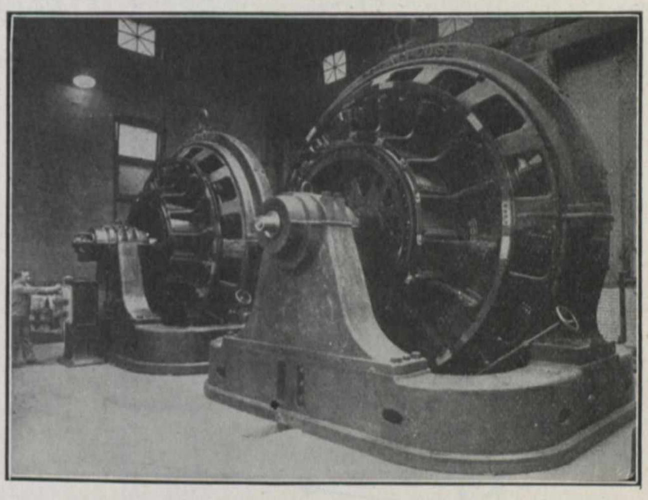
IRON ORES.

Iron ores are chiefly oxides, and native iron is almost unknown except in the meteorites occasionally picked up. Some native iron discovered by Nordenskjold at Ovifak, Greenland, in 1870, is now known to be of terrestrial origin, although it was at first supposed to be meteoric like other masses. A microscopic study of this iron has been lately made by C. Benedicks, who finds that it is really a natural steel high in sulphur and carbon, with a structure different from that of meteoric iron, and having the internal arrangement peculiar to carbon steel that has been rapidly cooled below 700 degrees C. This confirms the already preferred view that the Greenland iron—instead of coming to the surface in unoxidized condition—was reduced from the basalt by carbonaceous shale through which the molten mass was ejected.

3,000 KILOWATT ROTARY CONVERTERS.

For the economical transmission of energy as direct current either close proximity of the generator to the point of power application, or the use of alternating current for transmission is necessary. For this reason direct current for railway service is seldom generated as such in this progressive age. The tendency is to generate all energy as alternating current in large economically located stations, and to transmit it as alternating current to sub-stations located in the section where it is utilized as direct current in the railway motors. Rotary converters, because of their exceedingly high efficiency, are peculiarly adapted and are usually used in the sub-stations for converting from alternating to direct current.

It is recognized that the combined efficiency of a rotary converter and its transformers is considerably higher than that of a motor generator set of equivalent capacity and voltage. This advantage in efficiency is even more marked at light loads than at full load, and since the load factor of railway systems is usually low, the inherent fitness of the rotary converter is evident.



Two 3,000 kw. 25 Cycle Rotary Converters.

The growth of traction systems has been so rapid of late in the larger cities that it has been necessary to materially increase the outputs of the sub-stations. It was desirable, and practically necessary in some cases, to do this without increasing the sizes of the sub-stations to avoid increasing real estate investment. The Westinghouse 3,000 kw. rotary converter offers the solution to this problem in that with it maximum output can be obtained with minimum floor space.

A little over a year ago the Westinghouse Company built and installed two 3,000 kw., 25 cycle, 6 phase, 600 volt rotary converters in the sub-stations of the Interborough Rapid Transit Company, New York City, (Fig.) These two 3,000 kw. machines replaced two 1,500 kw. rotary converters. The 3,000 kw. machines occupy the same floor space as the smaller ones, but the output of the sub-station is doubled. Their success was so marked that seven more of the same type have been ordered by that company.

While some of these 3,000 kw. rotary converters are started from the direct current end, all are designed for alternating-current self starting and several will be regularly started by this method.

THE DESIGN OF RAILWAY BRIDGE ABUTMENTS*

By J. H. PRIOR, Assistant Engineer, Chicago, Milwaukee and St. Paul

The following paper gives the result of an investigation ordered by C. F. Loweth, chief engineer of the Chicago, Milwaukee & St. Paul, and made by the writer.

A railway bridge abutment is ordinarily a masonry structure which gives vertical support to one end of a steel

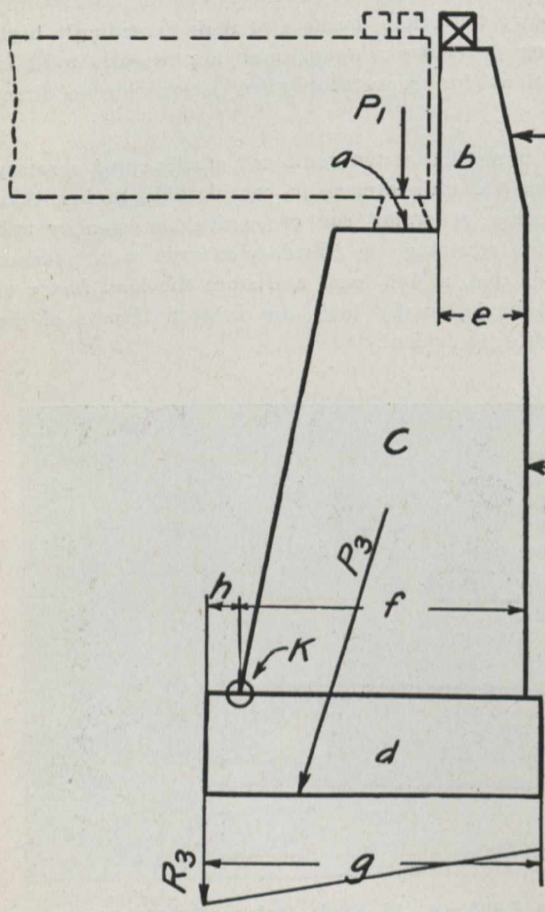


Fig. 1—Common Type of Abutment.

span, and at the same time gives whatever lateral support is necessary to prevent the adjoining embankment from slipping into the stream.

The most common type has a cross-section similar to that shown in Fig. 1, in which a is the bridge seat, consisting of a horizontal surface carrying the steel span; b is the back wall which supports the embankment and prevents its spilling forward on the bridge seat; the base e of the back wall b being made of such width as will make the back wall stable against overturning on account of the lateral pressure R-1 of the earth; c is the main body of the neatwork, and must have sufficient base f so that it will also be stable against overturning from the lateral pressure of the earthwork R-2; d is the footing, which must have a base g large enough to carry all the vertical loads, and offset h should be large enough to keep the pressure at the toe of the footing R-3 within the allowable limit.

The three principal types are the wing, U, and T abutments. In the wing abutment the wings keep the embankment from slipping into the stream. In the U abutment the wings are made parallel to the track, thus giving the lateral support to the embankment which is required to extend the embankment to the bridge seat. In the T abutment the floor is supported directly back of the bridge seat by the stem of the abutment, which carries the track back to a point where the embankment is of sufficient height to support it.

In addition to conforming to the ordinary laws of structural design, properly designed abutments should have the following properties, which may be called major requirements, because they affect the integrity of the structure:

m_1. The neatwork should be stable against overturning by revolving on the line k, Fig. 1, at the intersection of the front face of the neatwork and footing, and should also be safe against crushing on the same line.

m_2. The abutments should be stable against sliding, either by the neatwork sliding on the footing or the footing sliding upon the foundation bed.

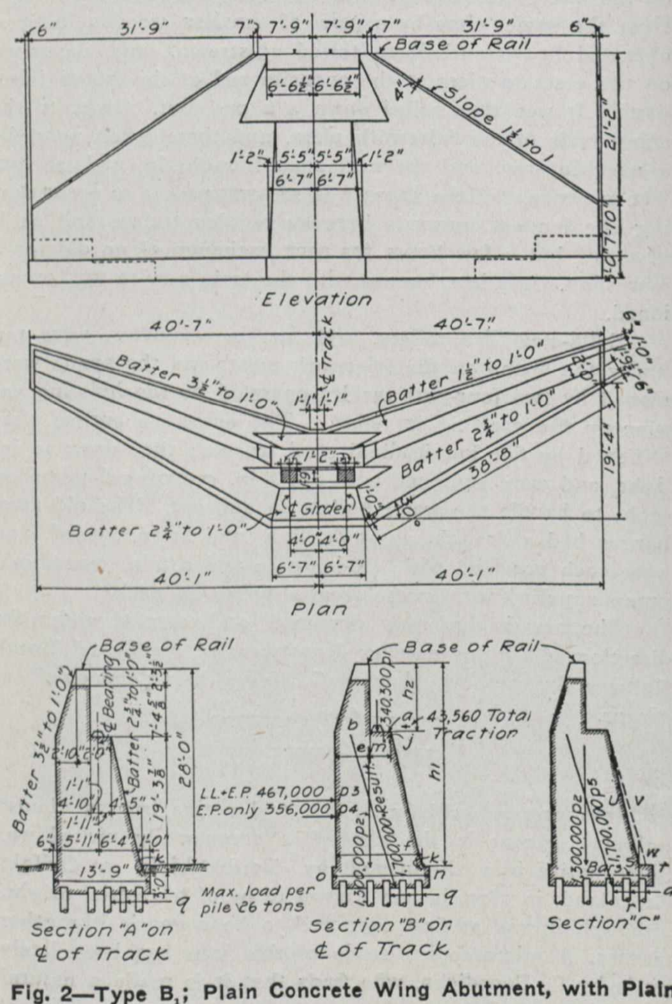


Fig. 2—Type B_1; Plain Concrete Wing Abutment, with Plain Concrete Backwall.

m_3. The pressure of the toe of the footing upon the foundation should not be excessive.

They should have the following properties, among others, which may be called minor requirements:

n_1. The abutments should protect the bank against scour.

*Abstracted from Bulletin 140 of the American Railway Engineering Association. Copyrighted by the association.

n_2. The abutments should prevent the embankment drainage from washing away the shoulder of the bank adjacent to the back wall.

n_3. The abutments should provide a joint which will support the track in an easy and continuous manner from embankment to superstructure.

n_4. The abutment should be easily drained.

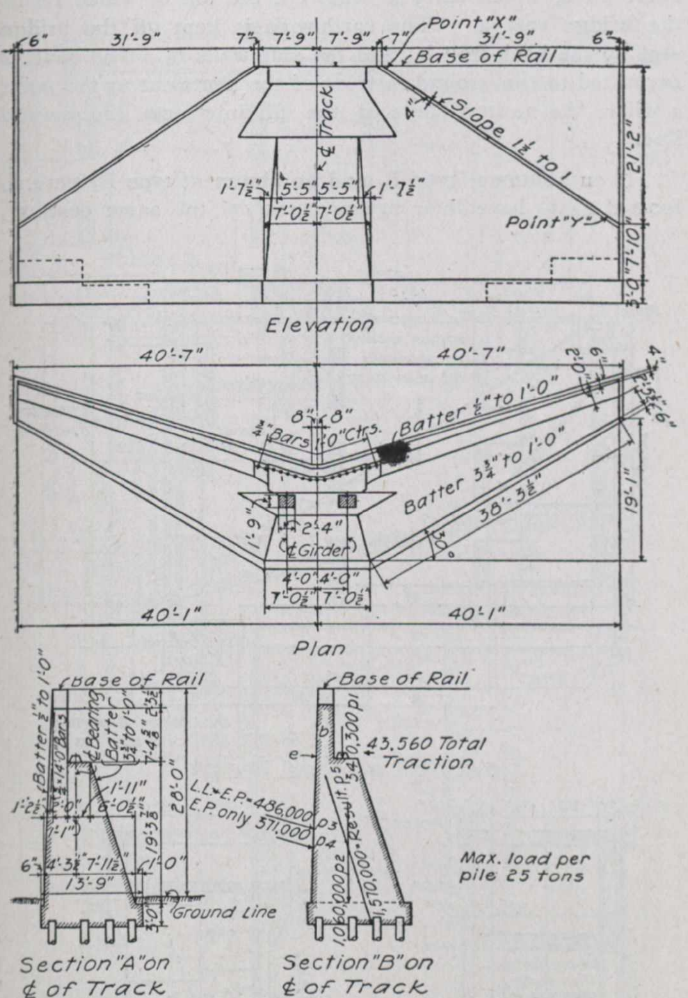


Fig. 3—Type B_2; Plain Concrete Wing Abutment with Reinforced Backwall.

As by far the greater number of abutments being built are either of plain or reinforced concrete, the term "Design of Bridge Abutments" at present means the design of bridge abutments in concrete masonry.

In order to compare the properties and economy of the various types, it was necessary to assume the same conditions in the design of all types. The more important assumptions which were made for this purpose are as follows:

(1) The height of the abutment is the distance from the base of rail to the natural ground.

(2) Slope of fill 1½ to 1.

(3) Slope of the natural ground away from stream or bridge opening 4 to 1.

Plain Concrete Abutments.

Type B_1.—The plain concrete masonry abutment without reinforcement in the back wall, type B_1, is shown in Fig. 2. The cross-section of the abutment, marked "section B on the centre line of track" is determined about as follows:

The distance h_2, from the base of rail to the top of the bridge seat a, is determined by the depth required by the floor, girders and bearings of the superstructure; and in new work the abutment must conform to these dimensions. The dimension e, which is the width of the back wall at its base, is taken as $4/10\ h_2$ if the lateral pressure of the earthwork tending to overturn the back wall is alone considered, or it is taken at $5/10\ h_2$ if a slight further provision is made for the overturning action of frost in the bank. The width of the bridge seat m is determined by the width required for the superstructure bearings plus whatever distance j is required to keep the bearings back far enough from the outer edge of the bridge seat to prevent the outside corner of the bridge seat from being sheared off in a diagonal direction.

The position of the ground line and the character of the foundation determine the distance h_1 from the base of rail to top of footing. If the base of the neatwork f is made $4/10$ of the distance h_1, the neatwork is stable against overturning or crushing of the line k, at the lower edge of the neatwork. The dimension f together with the width of bridge seat m and thickness of back wall e having been determined, we have the choice of setting the bridge seat and back wall vertically over the rear edge of the footing, or of locating the bridge seat and back wall nearer the front of the footing, thus decreasing

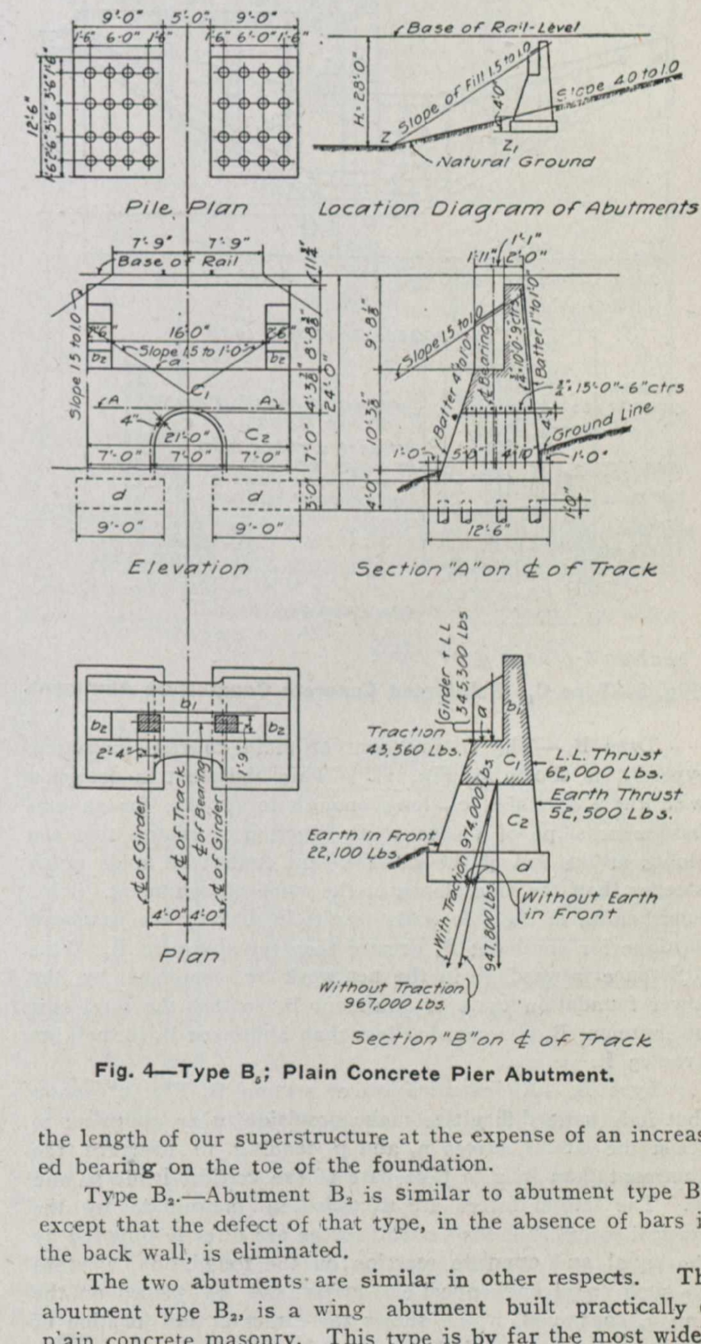


Fig. 4—Type B_3; Plain Concrete Pier Abutment.

the length of our superstructure at the expense of an increased bearing on the toe of the foundation.

Type B_2.—Abutment B_2 is similar to abutment type B_1, except that the defect of that type, in the absence of bars in the back wall, is eliminated.

The two abutments are similar in other respects. The abutment type B_2, is a wing abutment built practically of plain concrete masonry. This type is by far the most widely used, and is an economical abutment for certain heights.

Abutments of this type do not cause the drainage water to wash away the corner of the bank adjacent to the back wall at x in Fig. 3. but at the lower point y on the bank, where the amount of the drainage water is greater, there seems to be a tendency to wash the portion of the bank adjacent to the end of the wing wall, and, therefore, this point is often found protected by loose rip-rap.

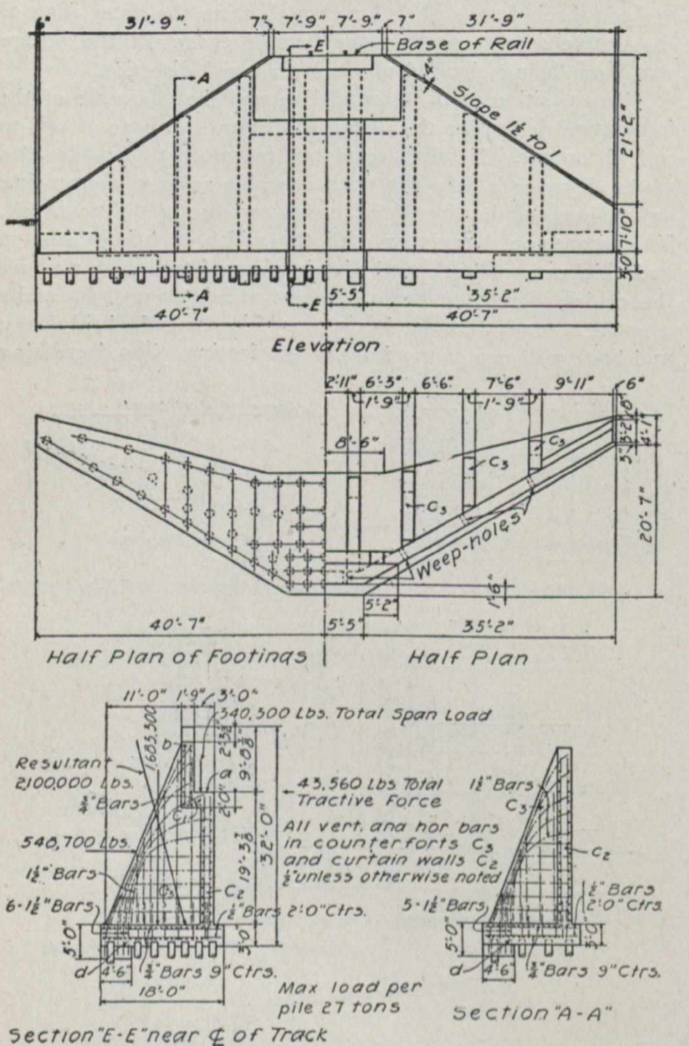


Fig. 5—Type C_1; Reinforced Concrete Counterfort Abutment.

Type B_3.—The U abutment of plain concrete masonry, type B_3, is shown in Fig. 11. This abutment is designed with the side walls, c_1, long enough to provide for an embankment slope of 1½ to 1, as experience shows that the slopes at the end of the embankment cannot be made much steeper than the side slopes of the same embankment. This lengthening of the side walls results in making the neatwork yardage for abutment B_3 greater than for abutment B_2. This difference in yardage in the neatwork is overcome by the lower foundation costs of abutment B_3 so that the total cost of abutment B_3 is somewhat less than abutment B_2 for heights over 23 ft.

Type B_5.—An examination of section B, Fig. 3, shows that it is more difficult to make provision in an abutment to resist the lateral forces p_3 and p_4 tending to overturn the abutment than it is to take care of the vertical loads p_1 and p_2. The lateral forces are exceeded in magnitude by the vertical forces, but the vertical forces are directly opposed by the equal and opposite reaction on the foundation, whereas no such equal and opposite reaction can be placed on the line p_3 continued, which shows the direction and location of the lateral pressure of the earthwork. As a consequence it would seem that a considerable economy could be effected in a design in which the lateral forces p_3 and p_4 were eliminated or greatly diminished. In abutment B_5, Fig. 4, little provision is made for resisting the lateral pressure of the earthwork. Instead, provision is made for diminishing the lateral pressure by omitting the wings and allowing the bank to spill around in front of the abutment. Abutment type B_5 consists, as shown in the elevation and Section B, of two short piers, which carry a beam c-l, the top of which forms the bridge seat a. The earthwork is kept off the bridge seat by the back wall b_1, and the side walls b_2. The bank is permitted to run around in front of the abutment to the point z where the natural slope of the fill intersects the ground line.

If an abutment type B_5 and an abutment type B_1 were so located as to have their bridge seats in the same position,

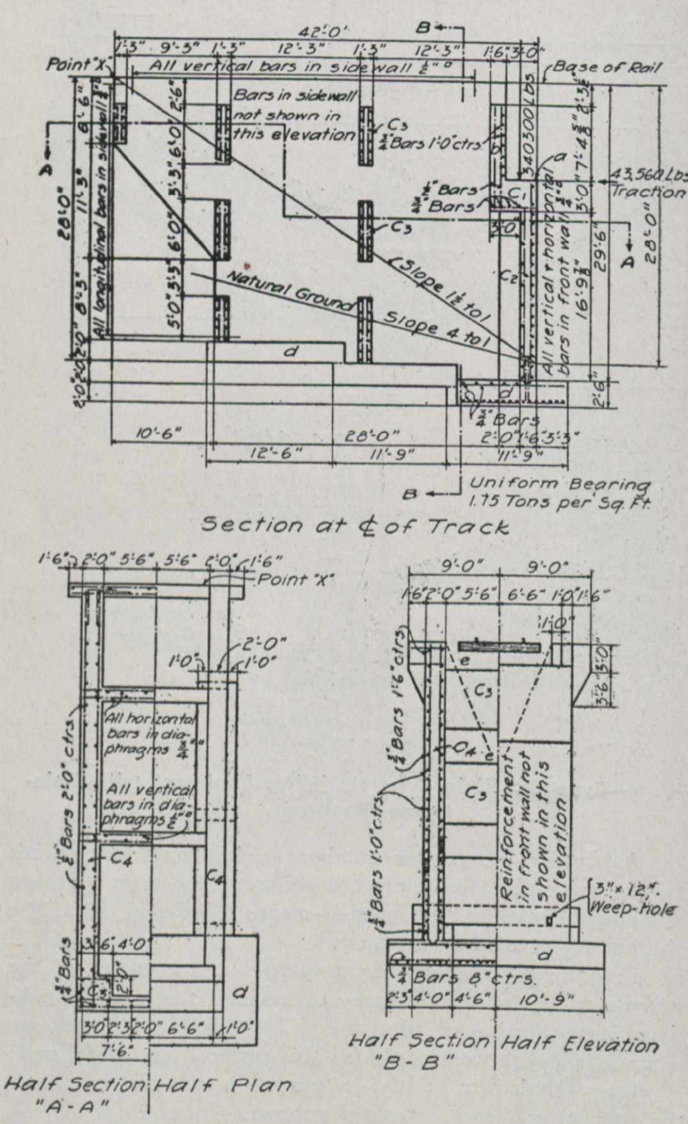


Fig. 6—Type C_2; Reinforced Concrete U Abutment Filled; Long Sidewalls.

the abutment B_5 would give a smaller waterway than the abutment type B_1 by the distance from z to z_1 in Fig. 4, to which the fill extends in front of the abutment. In order to get the abutment B_5 on the same basis for comparison as the other abutments, it is necessary to add to the cost of abutment B_5 the cost of extending the superstructure so as to permit the location of the abutment to be moved from z to z_1. As shown in the table herewith, this element of cost is an important item, amounting to 59 per cent. of the total.

Reinforced Concrete Abutments.

Type C_1.—The reinforced concrete wing abutment type C_1, shown in Fig. 5, is similar to the abutment type B_2 except that it is constructed of reinforced concrete instead of plain concrete. If the abutment is properly proportioned, the curtain wall c_2 can not be pushed outward by the lateral pressure of the earthwork without carrying with it the but-

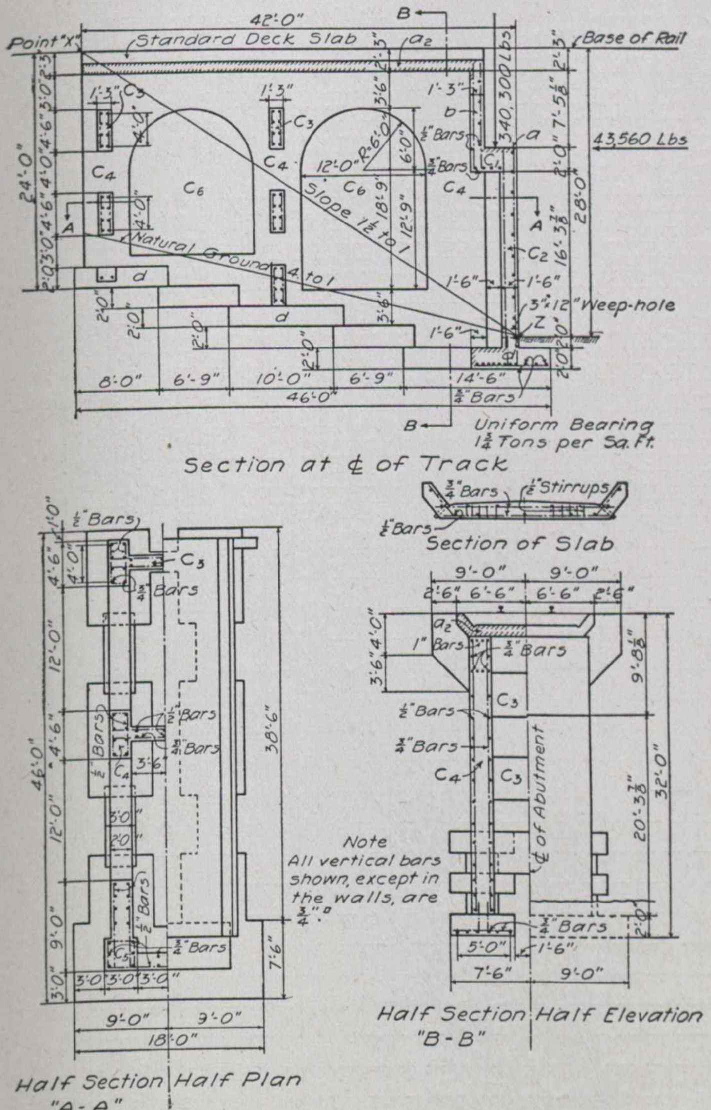


Fig. 7—Type C_4; Reinforced Concrete U Abutment, Covered.

tresses c_3 and without lifting the slab d from the pile foundations and also lifting the entire weight of the earth which rests vertically on the slab d. At the front of the abutment, the curtain wall c_2 is given a horizontal offset c_1 as shown in section E-E to provide for the bridge seat a. The curtain wall is carried up vertically from the bridge seat a, forming the back wall b shown in the same section.

The abutment shown in Fig. 5 has no particular advantage over type B_2, the introduction of buttresses not decreasing the cost of neatwork and the provisions for anchoring buttresses to the footings greatly increasing the cost of footings for fills above 20 ft. Fig. 5 does not, however, show the best design that can be made of reinforced concrete wing abutment. It contains two defects; the bridge seat is set too far forward and the footings have insufficient projection in front of the neatwork.

If these defects are corrected we have an improved and cheaper abutment. Estimates show that it costs about 10 per cent. less than type B_2 for the 36-ft. height and 25 per cent. less for the 50 ft. height which is outside the limits of the diagram.

If, in addition, the sections of all walls and buttresses are reduced to the minimum which can be placed by experienced workers under proper supervision, the economy over B_2 is still further increased.

Type C_2.—Abutment type C_2, Fig. 6, is a U abutment of reinforced concrete. It resembles the U abutment of plain concrete in external appearance. Structurally, however, it is quite different. It consists in plan of a rectangular box open at the top and at the embankment end, which is filled with earth. The track is directly supported on the earth filling. The sides of the box c_4 are prevented from being forced outward by the lateral pressure of the earth by ties c_3 which connect the two opposite sides c_4. The front of the box is the curtain wall c_2, which is a beam between the two

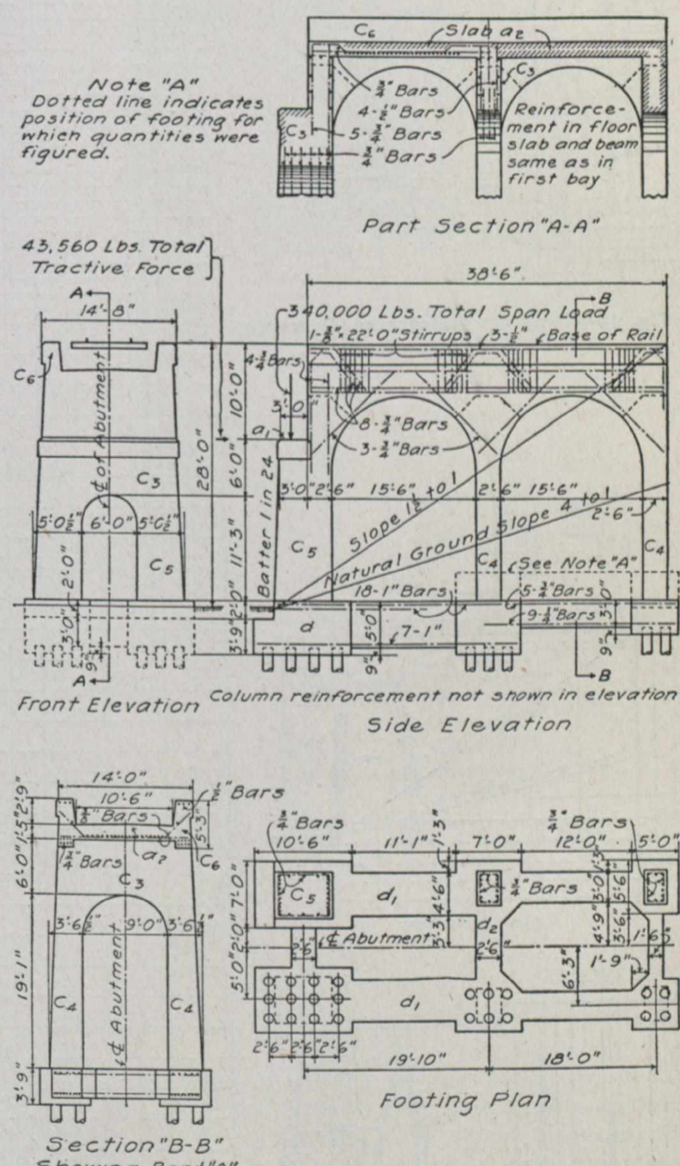


Fig. 8—Type C_6; Reinforced Concrete Arch Abutment.

side walls c_4, and restrains the lateral pressure of the earthwork in longitudinal direction.

This abutment easily satisfies requirements against overturning. The width of footings, measured along the center line of track, which is effective against overturning of type B_2, is 13 ft. 9 in. That of type C_1 is 18 ft. The corresponding dimension on abutment C_2 is the distance from the front to back of abutment or 45 ft.

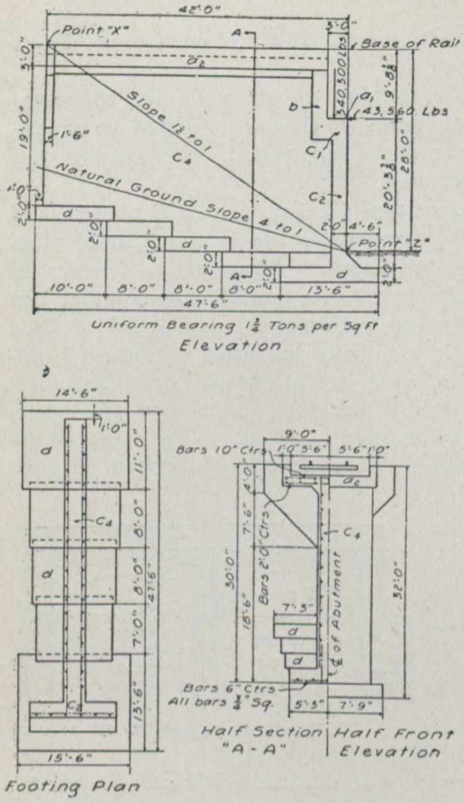


Fig. 10—Type C_6; Reinforced Concrete T Abutment.

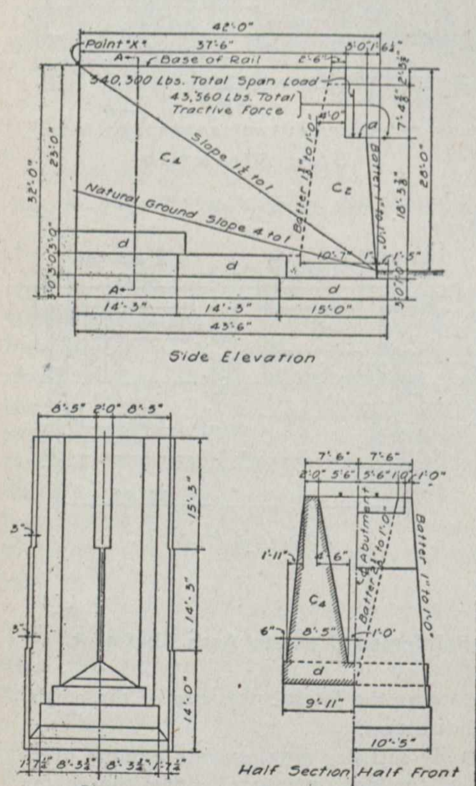


Fig. 11—Type B_3; Plain Concrete U Abutment, Filled.

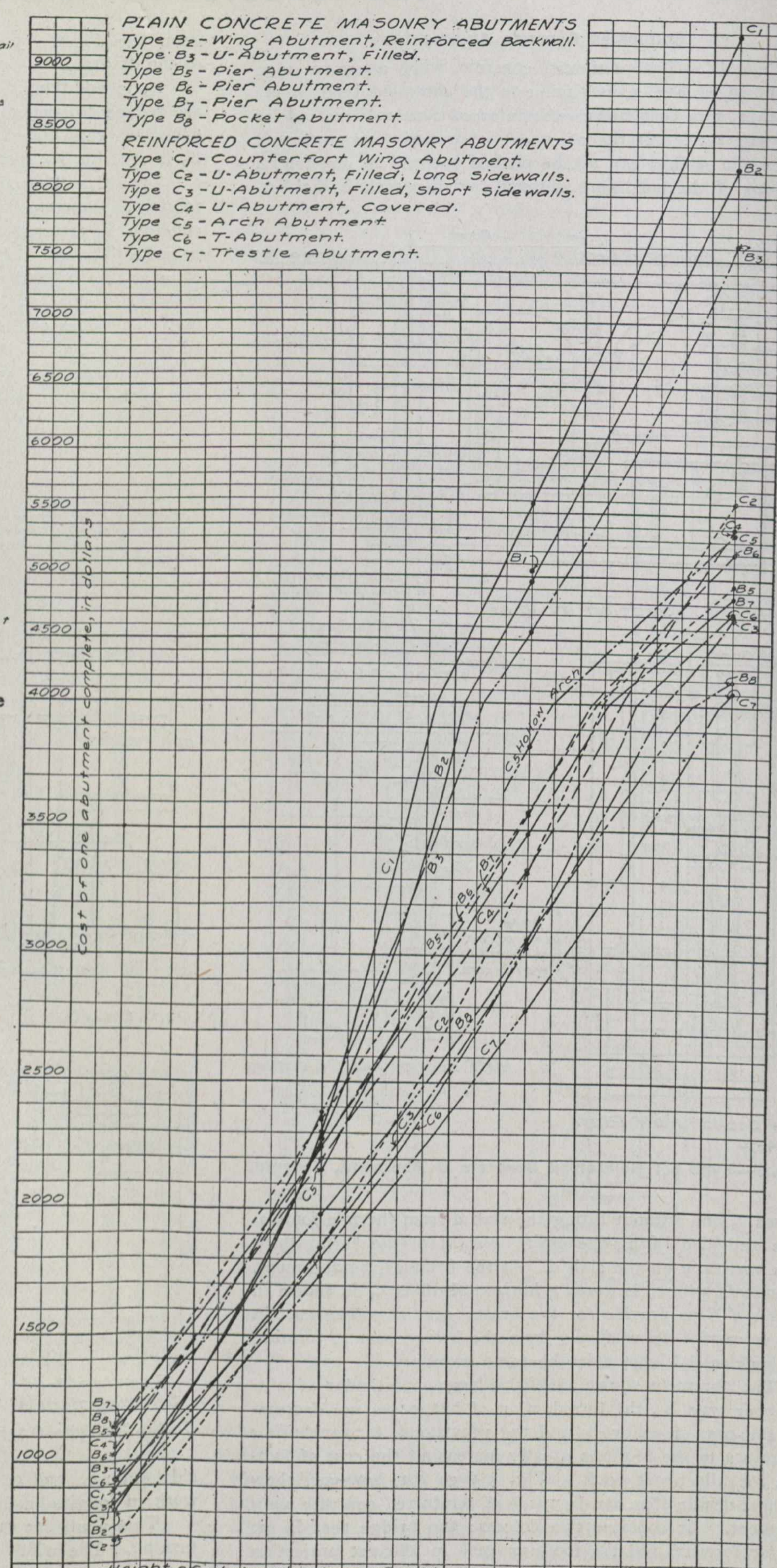


Fig. 9—Graphical Comparison of Costs.

It is true that the wing walls of abutments B_2 and C_1 protect, perhaps, five times as much of the embankment against scour as does abutment C_2, but if we compare the protection afforded by either type B_2 or C_2 with the total amount of bank protection required, the difference between the two abutments in this respect is not much. One of the defects of this abutment is that practically all of the surface water from the track for the entire length of abutment must drain past the point x, Fig. 6. As the amount of water is considerable, it tends to wash away the embankment at the end of the wing wall, which is, of course, an important matter, as the material is washed away at a point not far from the end of the track ties and requires much more attention than if the material was washed away at the end of the wing wall.

Type C_4.—Abutment type C_4, and all of the abutments which follow have one feature differing from any of the abutments previously mentioned, that of carrying the track directly on the body of the abutment over the entire length of the abutment instead of on the bank.

To save material and to equalize the pressure of the earthwork on both sides of the side walls c_4, two large openings c_6 are made in the walls. The portion of the side wall remaining between the openings c_6 forms a column; and the portion of the side walls above the openings forms a beam between columns. The openings are shown circular on top, although structurally they could have been as easily made square.

Abutment C_4 has an advantage over abutment C_2 in that its foundation loads are less, due to the fact that the interior of the abutment is only partially filled with earth. The side walls of C_4 are subjected to practically no unbalanced lateral pressure of the earthwork. It has the disadvantage that it requires a considerable amount of material for the construction of the floor slab a_2, and as a consequence it does not show much economy for low fills.

As there are practically no unbalanced lateral earthwork pressures in action against abutment C_4 no provision need be made to make it conform to the requirements m_1, and m_2 and m_3. This abutment gives, perhaps, slightly less protection to the embankment against scour than abutment C_2, but it will probably not wash away at the shoulder of the embankment at x, as drainage for the top of the abutment is provided through holes in the sides of the floor slabs.

Type C_5.—Abutment type C_5, consists in general of six vertical posts c_4 and c_5, which support the slab a_2. At the bottom they are tied together by the footings a_1 and d_2. The footings d_1 and d_2 act partly as foundation beams and partly as ties, which hold the bottoms of columns in their true relative position and afford them support against any unbalanced lateral pressure of the earthwork. At the top the cross beam c_3 spans transversely between the posts c_4. Two posts and the cross beam c_3 form a single bent.

At the top, also longitudinal beam c_6 spans longitudinally between the bents A. By this arrangement the slab a_2 is supported on four sides, the two ends of the slab resting on cross beams c_3, and the two sides of the slab are supported by longitudinal beam c_6.

At the front of the abutment the posts c_5 are made much heavier, as the beam c_3 which connects them together at the top carries the weight of the adjacent span of the superstructure, and also carries one end of a slab a_2. The divisions mentioned are only those which have to be made in order to execute the design; the structure itself being tied together by steel in all directions so as to resemble a monolith. In service this abutment has all the advantages and disadvantages of type C_4, which it resembles structurally in many respects. As it is somewhat more open than type C_4, it drains itself a little better. The abutment shown in Fig. 8 is the design of W. S. Lacher.

Type C_6.—A T abutment of reinforced concrete type C_6 is shown in Fig. 10. The stem of this abutment consists of a slab a_2, which is supported longitudinally along its center line by the central wall of the stem c_4. The wall is carried vertically down to the spread footings d.

The floor a_2 with the wall c_4 are given lateral stability against overturning by the front curtain wall c_2 and by the reinforcement on both faces of the wall c_4, which extends directly into the footings. The curtain wall c_2 is carried up to form the bridge seat c_1 and the back wall b in the same manner as in the C_4 abutments.

This abutment has the defect that it is not safe under a derailed locomotive. The derailed locomotive produces in

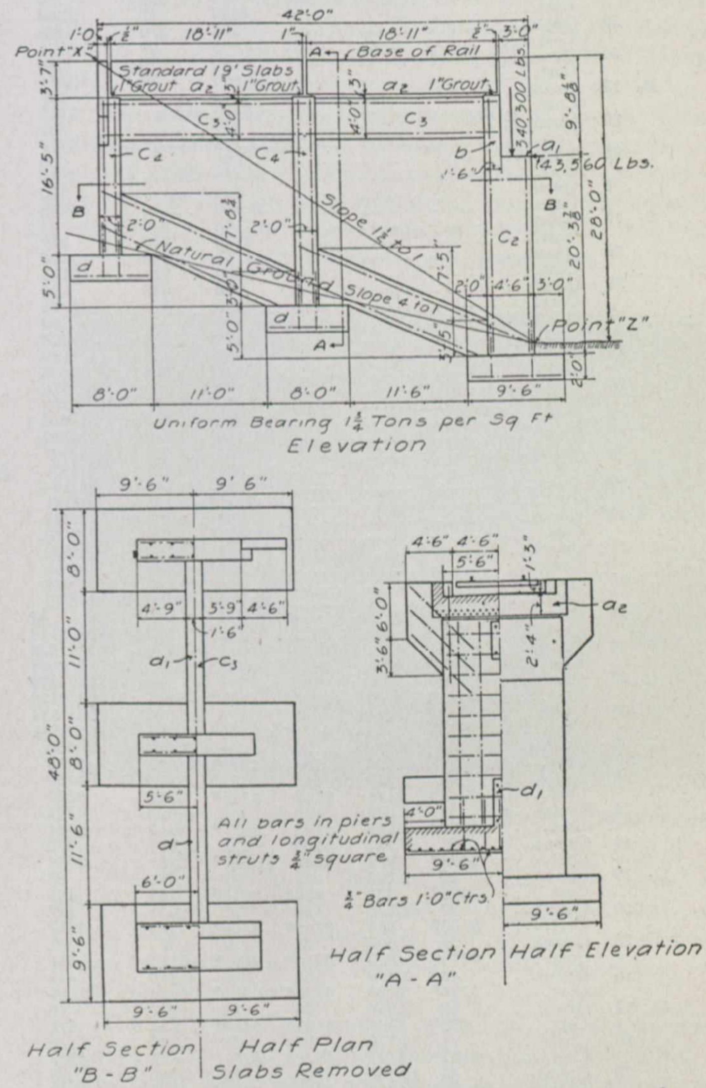


Fig. 12—Type C_7; Reinforced Concrete Trestle Abutment.

this structure much greater stresses than in the other types. If we increase the reinforcement to take care of the exceptional case of a derailment, we need a quantity of steel nearly double that shown in the table, increasing the cost of reinforcing steel practically $4,000 and filling the structure so full of bars that the cost of laying the concrete would be largely increased. Provision for derailment in this structure is, therefore, out of the question, and its weakness under a derailment must stand a grave defect.

Type C_7.—The trestle abutment type C_7, in a general way is a concrete trestle of sufficient length to carry the track from the point z, or from the end of the superstructure, to the point x, where the bank has attained its full height. Commencing at the top, this abutment consists of two standard U-shaped trestle slabs a_2 which contain the ballast, which in turn supports the ties and rails. These slabs rest on the neatwork of the bents c_2 and c_4, which are ordinarily reinforced concrete trestle bents with spread footings d. To resist unbalanced longitudinal pressure of earthwork, and also to add longitudinal stability to the abutment, the struts c_3 are introduced between the tops of the bents c_2 and c_4, and member and can be placed after the abutment has been built. The remaining portion of the abutment forms a complete structure without the slab. The slab is only used to afford direct vertical support to the track.

Memorandum on Total Costs of Various Types.

The table shows the manner in which the total estimated costs of the foregoing abutments were made up. This table

TABLE SHOWING METHOD OF CALCULATING COST OF ABUTMENTS.

Height in Ft.	Item.	Floor. Cu. Yds. of Conc.	Floor. Lbs. of Steel.	Floor. Sq.Yds. of Forms.	Neatwork. Cu. Yds. of Conc.	Neatwork. Lbs. of Steel.	Neatwork. Sq.Yds. of Forms.	Footings. Cu. Yds. of Conc.	Footings. Lbs. of Steel.	Footings. Sq. Yds. of Forms.	Cu. Yds. of Excavation.	Piles.	Cu. Yds. of Fill.	False-work.	Change in Length of Bridge one Abutment.	Engng. and Incidentals.	Grand Total.
B_1 28	Quant...	......	......	......	406	......	425	103	......	63	260	71	1,500	......	4.42 ft.	(5%)	......
	Cost....	......	......	......	$2,030	......	$510	$515	......	$76	$390	$568	$375	$50	$270	$239	$5,022
B_2 12	Quant..	......	......	......	60	270	121	21	......	23	65	......	170	......	.71 ft.	(5%)	......
	Cost....	......	......	......	$300	$9	$145	$105	......	$28	$98	......	$43	......	$43	$38	$809
20	Quant...	......	......	......	160	270	220	56	......	49	120	34	600	......	2.79 ft.	(5%)	......
	Cost....	......	......	......	$800	$9	$264	$280	......	$59	$180	$272	$150	......	$170	$109	$2,293
28	Quant...	......	......	......	387	270	420	102	......	60	260	64	1,500	......	6.04 ft.	(5%)	......
	Cost....	......	......	......	$1,935	$9	$504	$510	......	$72	$390	$512	$375	$50	$368	$236	$4,961
36	Quant...	......	......	......	696	270	628	142	......	75	400	108	3,100	......	8.38 ft.	(5%)	......
	Cost....	......	......	......	$3,480	$9	$754	$710	......	$90	$600	$864	$775	$50	$511	$392	$8,235
B_3 12	Quant...	......	......	......	74	......	125	31	......	......	60	......	60	......	......	(5%)	......
	Cost....	......	......	......	$370	......	$150	$155	......	......	$90	......	$15	$100	......	$44	$924
20	Quant...	......	......	......	229	......	285	59	......	......	115	......	250	......	.85 ft.	(5%)	......
	Cost....	......	......	......	$1,145	......	$342	$295	......	......	$173	......	$63	$200	$52	$113	$2,383
28	Quant...	......	......	......	496	......	450	103	......	......	210	......	525	......	1.52 ft.	(5%)	......
	Cost....	......	......	......	$2,480	......	$540	$515	......	......	$315	......	$131	$300	$93	$219	$4,593
36	Quant...	......	......	......	880	......	780	137	......	......	290	......	1,075	......	2.27 ft.	(5%)	......
	Cost....	......	......	......	$4,400	......	$936	$685	......	......	$435	......	$269	$400	$139	$363	$7,627
B_5 12	Quant...	......	......	......	26	540	64	26	......	......	.45	......	76	......	10 ft.	(5%)	......
	Cost....	......	......	......	$130	$19	$77	$130	......	......	$68	......	$19	......	$610	$53	$1,106
20	Quant...	......	......	......	52	540	96	25	......	......	48	27	400	......	22 ft.	(5%)	......
	Cost....	......	......	......	$260	$19	$115	$125	......	......	$72	$216	$100	......	$1,340	$112	$2,359
28	Quant...	......	......	......	80	945	134	25	......	......	46	32	960	......	34 ft.	(5%)	......
	Cost....	......	......	......	$400	$33	$161	$125	......	......	$69	$256	$240	$45	$2,075	$170	$3,574
36	Quant...	......	......	......	103	945	172	36	550	......	68	45	1,860	......	46 ft.	(5%)	......
	Cost....	......	......	......	$515	$33	$206	$180	$19	......	$102	$360	$465	$60	$2,800	$237	$4,977
C_1 12	Quant...	......	......	......	38	680	120	24	700	32	79	......	171	......	......	(12%)	......
	Cost....	......	......	......	$247*	$24	$144	$156*	$25	$38	$119	......	$43	......	......	$96	$892
20	Quant...	......	......	......	100	5,050	316	45	2,380	48	182	......	630	......	......	(12%)	......
	Cost....	......	......	......	$650*	$177	$379	$293*	$83	$58	$273	......	$158	......	......	$249	$2,320
28	Quant...	......	......	......	183	9,950	608	113	7,754	80	358	85	1,702	......	......	(12%)	......
	Cost....	......	......	......	$1,190*	$348	$730	$735*	$271	$96	$537	$680	$426	......	......	$602	$5,615
36	Quant...	......	......	......	306	16,600	1,007	176	12,070	86	602	141	3,450	......	......	(12%)	......
	Cost....	......	......	......	$1,989*	$581	$1,208	$1,144*	$422	$103	$903	$1,128	$862	......	......	$1,001	$9,341
C_3 12	Quant...	......	......	......	37	2,440	134	10	200	14	30	......	80	......	......	(8%)	......
	Cost....	......	......	......	$241*	$85	$161	$50	$7	$17	$45	......	$20	$10	......	$51	$687
20	Quant...	......	......	......	102	6,730	349	20	900	34	65	......	300	......	......	(8%)	......
	Cost....	......	......	......	$663*	$236	$419	$130*	$32	$41	$98	......	$75	$20	......	$137	$1,851
28	Quant...	......	......	......	189	12,130	602	37	1,180	41	130	......	630	......	......	(8%)	......
	Cost....	......	......	......	$1,229*	$425	$722	$241*	$41	$49	$195	......	$158	$30	......	$247	$3,337
36	Quant...	......	......	......	304	20,000	1,015	42	840	74	140	34	1,200	......	......	(8%)	......
	Cost....	......	......	......	$1,976*	$700	$1,218	$210	$29	$89	$210	$432	$300	$30	......	$416	$5,610
C_4 12	Quant...	8	2,180	48	31	1,950	25 99	16	400	24	60	......	70	......	......	(10%)	......
	Cost....	$52*	$76	$120*	$202*	$68	$63* $119	$80	$14	$29	$90	......	$18	$20	......	$95	$1,046
20	Quant...	15	4,100	81	69	4,340	45 224	31	2,110	47	100	......	200	......	......	(10%)	......
	Cost....	$98*	$144	$203*	$449*	$152	$113* $269	$202*	$74	$56	$150	......	$50	$40	......	$200	$2,200
28	Quant...	21	5,370	115	114	7,180	100 298	41	2,790	65	130	......	550	......	......	(10%)	......
	Cost....	$137*	$201	$288*	$741*	$251	$250* $358	$267*	$98	$78	$195	......	$138	$60	......	$306	$3,368
36	Quant...	27	7,360	148	175	11,000	125 534	68	1,500	94	220	62	1,050	......	......	(10%)	......
	Cost....	$176*	$258	$370*	$1,138*	$385	$313* $641	$340	$53	$113	$330	$496	$262	$60	......	$493	$5,428
C_5 20	Quant...	30	2,740	80	49	2,668	130	48	2,373	......	67	26	239	......	2.2 ft.	(10%)	......
	Cost....	$195*	$96	$96	$319*	$93	$156	$312*	$83	......	$101	$208	$60	$100	$140	$196	$2,155
28	Quant...	48	5,250	135	90	6,873	250	84	7,198	......	130	44	550	......	0.5 ft.	(10%)	......
	Cost....	$312*	$184	$162	$585*	$241	$300	$546*	$252	......	$195	$352	$138	$200	$30	$350	$3,847
36	Quant...	51	5,296	140	125	8,922	345	113	9,852	......	145	52	1,050	......	8.0 ft.	(10%)	......
	Cost....	$332*	$185	$168	$813*	$312	$414	$735*	$345	......	$218	$416	$262	$200	$490	$489	$5,379
44	Quant...	81	7,781	200	234	15,165	570	153	12,254	......	244	80	1,912	......	5.2 ft.	(10%)	......
	Cost....	$527*	$272	$240	$1,521*	$531	$684	$995*	$429	......	$366	$640	$478	$300	$320	$730	$8,033
C_6 12	Quant...	13	1,038	33	33	1,084	80	20	890	21	50	......	63	......	......	(10%)	......
	Cost....	$85*	$36	$83*	$215*	$38	$96	$130*	$31	$25	$75	......	$16	$10	......	$84	$924
20	Quant...	23	1,835	58	72	2,365	177	33	2,000	37	83	......	224	......	......	(10%)	......
	Cost....	$150*	$64	$145*	$468*	$83	$212	$215*	$70	$44	$125	......	$56	$20	......	$165	$1,817
28	Quant...	33	2,633	84	133	4,371	301	50	5,066	52	130	......	467	......	......	(10%)	......
	Cost....	$215*	$92	$210*	$865*	$153	$361	$325*	$177	$62	$195	......	$117	$30	......	$280	$3,082
36	Quant...	43	3,430	108	208	6,840	471	82	8,320	82	200	......	1,100	......	......	(10%)	......
	Cost....	$280*	$120	$270*	$1,352*	$239	$565	$533*	$291	$98	$300	......	$250	$30	......	$433	$4,761
C_7 12	Quant...	12	2,960	54	24	910	76	14	250	18	40	......	60	......	......	(8%)	......
	Cost....	$78*	$104	$135*	$156*	$32	$91	$70	$9	$22	$60	......	$15	......	......	$62	$834
20	Quant...	21	5,170	91	58	2,190	186	26	1,380	31	70	......	190	......	......	(8%)	......
	Cost....	$137*	$181	$228*	$377*	$77	$223	$169*	$48	$37	$105	......	$48	......	......	$130	$1,760
28	Quant...	41	10,100	129	87	3,290	283	36	1,910	37	100	......	500	......	......	(8%)	......
	Cost....	$267*	$354	$323*	$566*	$115	$340	$234*	$67	$44	$150	......	$125	......	......	$207	$2,792
36	Quant...	45	11,100	166	145	5,480	451	61	3,240	53	150	......	950	......	......	(8%)	......
	Cost....	$293*	$389	$415*	$943*	$192	$541	$397*	$113	$64	$225	......	$238	......	......	$305	$4,115

NOTE.—The unit prices used in calculating costs in the above table were as follows:
Concrete, $5.00 per yard, except in cases where the total cost is marked with an asterisk (*), where $6.50 per cu. yd. was used.
Steel, 3½ cents per pound.
Forms, $1.20 per sq. yd., except in cases marked with an asterisk (*), where $2.50 per sq. yd. was used.
Excavation, $1.50 per cu. yd.
Piles, $8 each.
Fill, 25 cents per cu. yd.

the struts d_1 introduced between the bottoms of the same bents. In addition to acting as struts these members c_3 and d_1 combine the three bents and the two spans into two rigid quadrilaterals. The trestle bent c_2 at the front end of the abutment is made considerably thicker than the bents c_4 in the bank in order to leave room for the bridge seat a_1. In service this abutment is much like C_4 and C_5. Structurally it resembles C_4 more closely in that the floor a_2 is a separate

should give dependable estimated costs, as the division of the total costs into their elements has been carried as far as could be conveniently done. It shows the quantities as well as the unit costs and makes it possible for the reader to substitute different unit costs for those shown, where the conditions are such that the unit costs given in the table would not apply. It is not believed, however, that any ordinary changes will effect the relations between the totals.

In Fig. 9 the total cost of abutments are platted as ordinates to the heights platted as abscisses.

The objection most frequently made to abutments types C_2 to C_7 is their high cost, especially, it is said, if constructed by men experienced mostly in the construction of plain concrete work. On account of this objection the principal unit costs for this type were taken rather high, as it was believed that there was enough economy in the design of some of these types to more than offset the highest unit cost which could be reasonably selected. Examination of Fig. 9 will show that this prediction was substantially correct.

In examining these cost curves, it is important that the following should be noted:

Types C_6 and C_7 are untried, and unexpected weaknesses may develop in their use. C_6 does not take care of derailment.

B_5 or C_7 cannot be used where much scour is anticipated or where the high water is near the bridge seat, without the use of rip-rap, whose cost has not been included.

It should be observed that the necessity of carrying the footings further below the ground will make a proportionately greater increased cost in types B_1 to B_5 than in types C_2 to C_7.

The use of any type of superstructure which gives a less depth of bridge seat below base of rail will mean a greater proportionate increase in cost in types B_1 to B_5 and C_6 than in types C_1 to C_5 and C_7.

Type C_1 is highest in cost of any abutment for heights over 21 ft. As previously mentioned, this type is created by the substitution of reinforced concrete in a mediocre design intended for plain concrete, making the least number of changes in the design which would permit the use of the new material.

If this design is improved, its cost can be reduced by an amount which will make it of less cost than type B_2 for heights above 28 ft. If advantage is taken of other known refinements in design of abutments of this character, its cost can be still further reduced.

For nearly all heights types C_3, C_6 and C_7 are the lowest in cost of those types in which the neatwork is carried to a sufficient depth to place footings on the natural ground.

Conclusion.

The writer finds that a general statement about the foregoing abutments has to be qualified in so many directions that it becomes merely a group of more or less disconnected facts.

Inspection, however, will show that the cheapest abutments for the higher fills, C_3, C_6 and C_7 are those in which no provision is made to restrain the lateral pressure of the earthwork, but where instead the earthwork is allowed to spill forward to its own natural slope.

As soon as departure is made from the gravity abutment, the greatest latitude is obtained for the ingenuity and skill of the designer. The types mentioned in this paper are only a few of the large number of abutment types which promise considerable economy.

As the minimum sections adopted are more liberal in the reinforced concrete abutments than in the plain abutments, it is probable that there is a wider margin for new economies in the reinforced than there is in the plain abutments.

THE WATAUGA POWER COMPANY'S HYDRO-ELECTRIC DEVELOPMENT.

By Francis R. Weller, Civil Hydraulic Engineer, Washington, D.C.

There is now nearing completion on the Watauga River, at a place known as "The Horse-shoe," about six miles above the town of Elizabethton, Tenn., a water power development, which, when completed, is expected to play an important part in the economic development of the northeastern sections of Tennessee.

The Watauga River rises on the northeastern slopes of the Grandfather Mountains in Watauga County, North Carolina, and flows through Watauga County, N.C., and Johnson and Carter counties, Tenn., discharging into the Southern fork of the Holston River, about nine miles west of Johnson City.

The river at the site of the development emerges from a gorge through the Holston Mountain. At this point the fall is quite great, and all natural conditions are extremely favorable for the location of a high dam. Immediately below the dam site, the valley widens out, which enables the flood waters to be carried off without any great rise in the river.

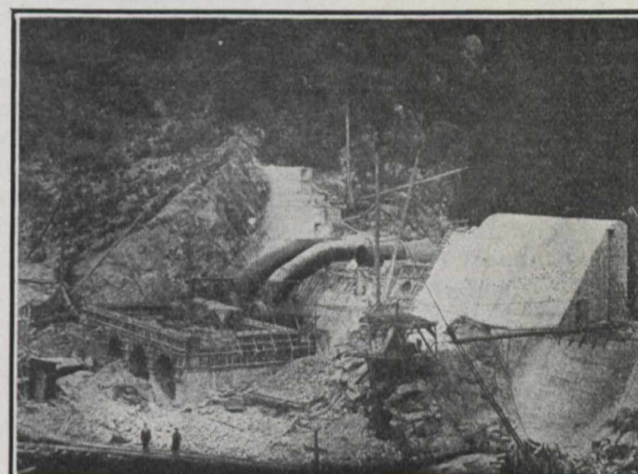


Fig. 1.—The Power House and Dam During Construction.

The site selected for the dam is an ideal one, rocky precipitous slopes affording excellent anchorage for each end of the dam. The rock formation is of granite. The surface of the rock is rough and irregular, which has given a very good bond to the dam.

The Watauga River and its tributaries are fed by many springs, which tend to yield a uniform flow. The utility of a stream for water power purposes is dependent largely upon the character of its stream flow. Streams that are subject to very low periods of flow must be supplemented by steam power, which naturally, involves additional expense and lessens the value of such streams for water power purposes.

Droughts are infrequent in the Watauga Valley, and only of short duration. The drainage area of the stream is in the region of maximum rainfall, and the headwaters are in the proposed Appalachian Forest Reserve, which the United States Government has created for the purpose of aiding navigation, and incidentally the improvement of water power by reserving the forests on the water-shed of southern navigable streams. This forest reserve will undoubtedly prove to be of great value for all southern water power developments, for it is a demonstrated fact that forests tend to maintain stream flow by retarding and regulating the rainfall, and conserving the springs, which are the sources of nearly all mountain streams.

Hydro-electric power, when economically developed, is much cheaper than steam power. A community, therefore, which can obtain hydro-electric power possesses a valuable asset, which will permit the rapid growth of the manufacturing industries.

The use of hydro-electric power in recent years has become so extensive that manufacturers now realize the advantage of purchasing their power instead of investing large amounts of their capital in steam plants, which will only cause many annoyances and additional expense.

The officers of the Watauga Power Company realized the demands for hydro-electric power in the rapidly growing cities of Bristol, Johnson City, and Elizabethton, and in the spring of 1910 organized the Watauga Power Company, which acquired two water power sites on the Watauga River, also one on the Doe River near Hampton, and also purchased the small existing hydro-electric plant now supplying light to the town of Elizabethton. The company immediately took steps toward the development of the first power on the Watauga River. But preliminary to actual construction, they made a contract with the Bristol Gas & Electric Company for the sale of a large amount of the power to that company, which will in turn distribute the power to the various industries in Bristol.

Certain amount of the power was reserved for the needs of Elizabethton, and the surplus power will be either sold in Johnson City or in Bristol as the growing needs of the cities require it.

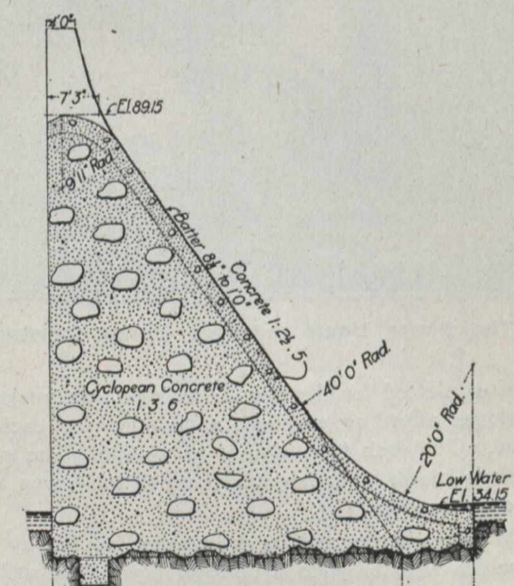


Fig. 2.—Cross Section Through Dam.

Contract was awarded to William J. Oliver, of Knoxville, Tenn., for the construction of the dam, power house, and tail-race, and for rebuilding the Virginia and Southwestern Railroad Bridge above the dam site; also the Westinghouse Electric & Manufacturing Company, of Pittsburg, Pa., received the contract for the electric apparatus and equipment, and the S. Morgan Smith Company, of York, Pa., the contract for the hydraulic machinery, and the National Electrical Supply Company, of Washington, D.C., the contract for the construction of the transmission lines. Work on the dam was begun July 1, 1910.

The dam is of solid cyclopean concrete, gravity type, 55 feet high above low water, 7 feet 3 inches wide at crest, and 58 feet 6 inches at base. It is designed to carry the maximum discharge of the river, which is equal to a flow of 12 feet in depth over the crest of the dam. The length of the spillway section of the dam is 240 feet, and at the north end of the spillway the abutment section is 12 feet high. This abutment section contains headracks and gates, which control the flow of water through three 7-foot 6-inch penstocks, delivering the water to the turbines. The turbines are of the reaction type, each unit consisting of two 33-inch horizontal shaft wheels running at 360 R.P.M., with a guaranteed efficiency of 80 per cent. when operated at full load. The mechanical power on the shaft of each of these turbines at full gate is 1,600 horsepower.

These turbines are located outside the power house and immediately behind the head wall or abutment. The shafts of the turbines extend through the power house wall, and on each shaft is direct connected one 1,250 K. V. A. 2,300-volt alternator. Each generator is separately excited by means of exciters mounted on the end of the generator shaft. The generator voltage is raised to 44,000 volts for line transmission by means of three single phase step-up transformers. Immediately outside of the building will be located electrolytic lightning arresters.

The power house is of ornamental design supported on heavy concrete piers and arches. The water, after leaving the wheels, will be discharged through these arches into the tail-race, excavated out of solid rock, and approximately 250 feet long.

The super-structure of the power house is constructed of reinforced concrete and absolutely fire-proof. All doors and window sashes are of steel.

In the power house will be installed a twelve-ton travelling crane for the purpose of handling machinery.

Only two generating units, a total of 3,200-H.P., will be installed at this time. Space has been provided for the third unit, which will be installed in the future when the demands for additional power are made.

The backwater caused by the dam will extend up the river about 1½ miles, and will form a pond with a surface area of about 3,700,000 sq. ft.

It was found necessary to raise the present railroad bridge five feet. This involved rebuilding all piers abutment and eliminating present trestles by means of an earth fill and a plate girder. All of this work has been done and traffic maintained without interruption.

The transmission line for the first five miles from the power house crosses one of the spurs of the Holston Mountain, and it is practically an air line to Bristol.

The conductors are No. 3 stranded aluminum wire mounted on 50,000-volt insulators. The line is substantially built, and every possible care was taken to prevent any future break-downs, which might cause interruption to the service.

The company has arranged for sub-stations in Bristol and Elizabethton, where the voltage will be reduced to 2,300 volts for distribution.

The officers of the company are as follows:

Lee F. Miller, President.
Walter E. Hunter, Vice-President.
J. H. Grayson, Secretary and General Manager.

All of these gentlemen are residents of Elizabethton.

The plans were prepared and the construction carried out under the direction of the writer. Mr. R. L. Weide has been resident engineer on the work since its beginning, and Mr. M. A. Weller, assistant engineer.

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CONTENTS OF THIS ISSUE.

TENDERS FOR TORONTO TUBES.

In these columns we have always held that absolute fair dealing with British and foreign tenderers for Canadian work should be the rule. Sometimes, it must be said, this ideal is not reached; and it would appear that British tenderers are not given an equal chance. After the tenders for the Toronto tubes were closed, certain papers took issue with the publicity given to the work in Great Britain. We feel that these papers did not have all the facts in the case, and that, therefore, their view of the situation was a little distorted.

While it must be admitted that in many cases in the past Canadian municipal authorities have failed in this matter of publicity of contemplated work, still in this case, to one cognizant, it must be admitted that adequate publicity for the work was given in Great Britain.

Over a year ago the British Trade Commissioner in Toronto reported to London that this work of subway construction was being contemplated, and at frequent intervals since that time the public of the Old Country have been apprised of all new developments in the situation. When conditions had arrived at the point when it would soon be time to call for tenders, the officials at the City Hall forwarded this information to the different technical papers in England. In the issues of September 8th, 15th and 22nd the "Surveyor," "Engineering," and "The Engineer" contained preliminary announcements stating that on November 1st tenders would be closed for the contract for building three miles of subway. Later, on October 13th and 20th, these same papers carried the advertisement calling for the tenders. Now, we contend that under these circumstances adequate publicity was given for this work. As a matter of fact, we do not believe that British contracting firms were very much interested in this work, for the contract dealt only, or, at least, nearly altogether, with labor. If it had been a question of furnishing equipment, rails, etc., we feel that there would have been a number of British tenders. This, no doubt, accounts for the fact that so few British contractors were interested.

We feel that this explanation is only fair to the city officials of Toronto, for their attitude throughout has been to give the very widest measure of publicity to this work, and we feel, therefore, that the statements which have appeared in the press have been very unfair to them.

TESTING A LARGE GENERATOR.

At the McCall Ferry power plant a test was recently made upon a 7,500 k.w. generator. The unit tested consists of two runners on a vertical shaft, on the upper end of which the rotating field of the generator is mounted. The generator was operated as a motor while connected with first one and then both of the hydraulic runners.

The test was made by connecting the armature of the generator in series with the armature of another generator of the same size. Thus, when the second unit was started, the current passing into the generator under test caused it to speed up as a motor. When they were brought up to above speed the two machines were cut apart electrically, and the unit under test was allowed to revolve until stopped by friction, windage, and other losses. During the time of retardation speed readings were taken every two seconds.

With the above data secured the losses were calculated in the usual way. This method of retardation test is a valuable one, and it is of interest particularly when used in connection with such a large generator. It allows, too, of securing data with respect to certain losses in the hydraulic runners as well.

FUNDS FOR McGILL.

McGill University are again having trouble with their finances. It seems decidedly unfair that a university such as McGill, which has turned out so many admirable men, should be handicapped and their work curtailed by the fact that they cannot get financial support to keep up their work. For the past few years the university's expenditures have exceeded their income by from $30,000 to $65,000 a year. Private donations in the past have aided the university materially, and, in fact, have placed it in its present magnificent position. However, the work of the university is work which must be supported lavishly by the government. We find, however, that Quebec pays only about three thousand dollars per year in its support. When we look at what Ontario is doing for Toronto University, it appears that Quebec is neglecting its duty with reference toward higher education.

The university authorities have recently approached the Quebec legislature to endeavor to secure a more liberal grant, and there is a movement among the graduates to raise a million dollars to aid the work of the university.

When we appreciate the position held by the Medical and Engineering Faculties of McGill at the present time, it seems too bad that their enviable position must be sacrificed unless more adequate support is given them. We extend our hearty good wishes to McGill in their coming campaign to raise one million dollars.

A SCHOLARSHIP AT THE UNIVERSITY OF TORONTO.

The Department of Mechanical Engineering of the University of Toronto has been made the recipient of a scholarship of the value of the fees of the fourth year through the generosity of the Board of Inspection and Insurance Company of Canada. The award is to be made to the student who stands first in honors in the work of the third year in that department. This is the first scholarship given to the Faculty of Applied Science of the University of Toronto with the exception of the two research scholarships granted by the Engineering Alumni Association as a token of the confidence of the graduates in the work of their Alma Mater.

The scholarship just granted must be a source of great satisfaction to the university, and to the Faculty of Applied Science in particular, that this company should in such a practical manner show its interest and its desire to stimulate research in this branch of the work of the university.

There is evidently a new interest springing up in connection with the work in Applied Science and Engineering. The industries are beginning to appreciate the benefits to be derived by a closer association with the teaching body. It is to be hoped that this interest will increase, for, by closer and more intimate contact here, much good will develop to both the industries and the faculties.

EDITORIAL COMMENT.

If Canada would give ear to proper city planning schemes, many complex problems for a later day might be saved. With all the land we have, houses are being built on small lots, close together, in cramped streets, in a manner which is making surely trouble for the future.

* * * *

The chairman of the Royal Commission of Technical Education, Dr. J. W. Robertson, says in a statement of his impressions of Ireland that they were able to learn a great deal concerning efficiency in organization and in the training of the workers there. The organization of instruction for the rural population in Ireland provides for all classes in all localities. A new spirit of confidence and hopefulness seems to be replacing the old attitude of resentment at conditions there.

* * * *

Some of the publications interested in lumber and stone in the United States have been making biased statements relative to the strength of concrete structures —statements which will only tend to destroy public confidence in the numerous structures throughout the country. Too much of this sort of criticism is made: those interested in one type of construction crying down another's wares, and this spirit will only result in injury to all. It should be the aim of all to endeavor to find out for what particular kind of work each material is fitted for, and use it for that purpose. For that reason there should be the heartiest sympathy and co-operation between all.

BUILDING STONES IN ONTARIO.

While a considerable amount of fairly good building stone will continue to be produced from the different formations throughout Ontario, Professor W. A. Parks, in a report to the Mines Branch, Ottawa, is of the opinion that, we must look to a development of the crystalline limestones, marbles, and granites, for a regeneration of the stone industry. Concrete is taking the place of stone for all heavy construction, and is rapidly replacing it for the cheaper types of building. With increase of wealth, the highest grades of stone are demanded for buildings of a monumental character. Most of our sandstones and limestones fall short of this standard, but when more is known of the crystalline limestones and marbles they will be more largely employed for structures of the highest type.

In the erection of fine buildings in the cities, there is a growing tendency towards the use of granite, which is likewise replacing limestone for monument bases. Modern quarrying methods and machinery have largely reduced the difference in cost of quarrying and working granite, as compared with the softer stones. In consequence, we may confidently expect to see a steadily increasing demand for granite. This demand can not be met either by the quality or the amount of granite at present produced in Ontario. Is it not a reasonable assumption that some at least of the numerous deposits of granite in the province will be able to supply the demands of the future?

It cannot be denied that there is a serious decline in the production of building stone. The chief cause for this decline, and the reasons for the present condition of the industry are tabulated below:—

(1) The use of cement for heavy construction, such as the building of bridges and canals.

(2) The use of cement blocks and artificial stone for architectural purposes.

(3) The cheap importation of Indiana limestone, and Ohio sandstone.

(4) The modern custom of erecting steel buildings, and facing them with terra-cotta, glazed brick, or artificial stone.

(5) The failure of the Medina brown stone.

(6) The increasing demand for granite, to which Ontario has failed to respond.

(7) The high wages demanded by stone-cutters, and the difficulty of procuring a sufficient number of competent men.

(8) The fact that most of the stone quarries are in the hands of very small operators, who work them only on receiving an order. In consequence, there is always a delay in delivery, and stone of a mixed character is shipped. These same owners do not devote their time to the stone business; it is merely incidental, hence receives little attention. If there were more strong companies actively and aggressively carrying on stone quarrying as a business, the cost of stone would be reduced, and the use of concrete, for architectural purposes, restricted.

THE GOOD AND BAD OF COAL SPECIFICATIONS.

That the success of coal buying and selling on specifications depends on the reasonableness of the specifications themselves, is an important matter discussed in a recent paper by Dwight T. Randall, of Arthur D. Little, Inc., Chemists and Engineers, of Boston. He says on this head:

After it has been decided what kind of coals may be burned successfully in any given plant, it is important that the specifications be so drawn that it will be to the interest of the dealer to deliver the kind of coal which has been established as standard in his proposal and prevent the substitution of lower grades of coal which might be difficult to burn with good results. It is evident that a specification based on heating value alone will not do this, and that there should be some clause making it possible to reject the coal, or to burn it and pay for it at a reduction in price greater than that due to B.t.u. only.

There has been a great deal said for and against the plan of purchasing coal on a guaranteed analysis and, as is often the case, both sides are right, but they are really discussing different things.

A properly drawn specification protects the dealer who is prepared to furnish good coal in competition with dealers handling inferior coals at the same price. Where these specifications permit the coal contractor to state the analysis of his coal which on acceptance of the bid becomes the standard for the contract, there need be but little variation in price if the dealer is familiar with the analysis of the coal offered, and if the standard is based on average values the premiums and penalties for the year should practically balance each other. Many dealers have bid on impossible analyses and then blamed the specification plan for their losses.

A properly drawn specification, providing for premiums for better coal than specified, encourages the coal operators to exercise greater care in mining and in picking the coal before shipping, and enables them to secure a return on the cost of such preparation. Most consumers have found that it is not profitable to pay freight on an unnecessary amount of slate and ash in the coal.

The important items in a specification are as follows:

(a) A statement of the amount and character of the coal desired.

(b) A statement regarding the conditions for delivery of coal.

(c) A statement regarding the disposition which will be made of the coal in case it is outside the limits specified.

(d) A statement regarding the corrections in price for variations in heating value, for variations in ash and for variations in sulphur, provided it is found advisable to limit the percentages of ash and sulphur in the coal to be delivered.

(e) A blank form on which the dealer may submit price and the kind and quality of coal which he proposes to furnish.

There are several forms which have been prepared along these lines which have proved satisfactory. It is necessary in almost every case to modify the specifications to fit the special conditions in the plant and the fuels which are available.

In the past many dealers not familiar with the quality of the coals have bid on contracts and guaranteed a quality of coal that was better than can be delivered from any mine in the United States. Naturally these analyses that had been useful as exhibits were found to be poor standards on which to base the guarantees of coal to be delivered. Many progressive dealers have recognized the reasonableness of the demand for a standard for quality of the coal to be delivered and they are selling coal on a basis which secures for them the average price they expect to get for the coal.

The importance of testing coal purchased under contract may be illustrated by two recent cases. In Case 1 the coal was guaranteed to be Georges Creek, and in Case 2 to be New River (Table 3).

Table 3.—Data of Coal Purchased Under Contract.

Case 1—

	Guaranteed Analysis as Delivered.	Coal Delivered.
Ash in dry coal..........	8.00	12.66
B.t.u. in dry coal........	14,250	13,558

Case 2—

	Guaranteed Analysis as Delivered.	Coal Delivered.
Ash in dry coal.........	6.00	8.48
B.t.u. in dry coal........	14,700	13,981

Neither of the parties in the above cases had made a practice of having the coal which was delivered at their plants sampled and analyzed. In such cases the blame for paying a good price for a poor coal rests with the purchaser. The dealer probably knew as little about the quality as the man who paid for it.

The plan of purchasing coal on a specification based on a guaranteed analysis may or may not be a good one, depending on circumstances. It has proved satisfactory in nearly every case when the specifications were so drawn as to protect the buyer against substitution of other coals, and at the same time was perfectly fair to both the buyer and to the seller. Such specifications actually protect the dealer against unfair competition, as has been shown in many cases. This plan will not be satisfactory if the specification is carelessly drawn and the coal is selected on the basis of price without regard to its adaptability to the furnaces; nor will it prove satisfactory if the sampling is done by ignorant or careless men and the analysis made in poorly equipped laboratories by inexperienced chemists. Whether it will be advisable to purchase for a plant on this basis depends on the amount of coal used, the amount delivered at one time, the kinds of coal available, and whether the purchaser and the dealer are qualified by a knowledge of the available coals to enter into a contract on this plan.

Metallurgical Comment

T. R. LOUDON, B.A.Sc.

Correspondence and Discussion Invited

IRON INDUSTRY OF CHILI.*

It is only twenty-five years ago that people commenced prospecting for iron mines in Chili. In 1905 a French company was formed for the starting of an iron industry in South Chili. Engineers who have had the opportunity of visiting the principal iron-ore deposits in the United States and in Europe have affirmed that outside Brazil and some other regions—such as Algeria and Morocco, for instance—no other country offers iron-ore resources approaching in quantity and quality those of Chili.

The principal iron-ore deposits in Chili belong to the same geological formation. They are broadly divisible into two classes—the pure iron-ore deposits and those mixed or combined with copper-bearing minerals, or transforming them selves at greater depths into minerals more or less cupreous. The ores are found in lodes and beds, often very thick, and sometimes in large irregular deposits which form massive mountains of high-grade iron ore. Some of these cupreous iron beds, as in the Department of Chanaral (Pueblo Hundido, Carmen, Tres Gracias, etc.), constitute real copper mines, yielding very important outputs of this latter metal. In the north of the province of Tarapaca, near the Cordillera, iron ore has been found, generally in thin lodes and combined with other minerals. Near Mejillones bay, a few miles distant from the coast, are important deposits of iron ore, which are generally cupreous and combined with manganese. In the whole of the province of Antofagasta, lodes and beds of iron ore are found, often mixed with copper. Near the harbor of Taltal, above the small bay of Hueso Parado, very important deposits of ore have been found, and are now exploited. The mineral is of high grade, often pure (in the district called Norte Magnetico), but also in certain places rather phosphoric or a little cupreous. These deposits, lying at the top of a mountain, almost perpendicularly, could be easily and economically exploited. The transport and shipment would not present any difficulty. Some of the deposits are magnetite, others hematite.

A few miles from the coast, in this same district of Taltal, rather important deposits of hematite are visible; but it is the provinces of Macama and Coquimbo which contain the most important iron-ore beds. In the Chanaral district, near Pueblo Hundido, and near Inca, are found the large cupreous iron-ore deposits of which mention is made above, and which are exploited for copper. Near Carrizal there are high-grade and very pure iron-ore deposits close to manganese-ore deposits (Coquimbana, Negra) of the district. In the Copiapo Department, near Tierra Amarilla, important deposits of iron have been discovered, which it is expected will later be easily exploited and carried down by the existing railway to the port of Caldera, a first-class harbor.

The Vallenar Department is, so far as at present ascertained, that which contains the most numerous and most important iron-ore deposits. A few miles from the town of Vallenar numerous outcrops of iron ore appear (on the plateau overlooking the town) which have been exploited for use as flux in blast furnaces for smelting siliceous copper and silver minerals. In the south-east are the deposits of the mountain "Ojos de Agua." About 8½ miles further south-east, near the mountains "Perdices" and "Rosario," and about 39 miles distant from the coast, appear the remarkable deposits of Algarobo and Algarobillo, concessions for which have been granted to a French syndicate. Further south, in the desert, other deposits exist. This ore occurs either pure or mixed with copper, as is the case at Cristales and Ratones, but at present transport to the coast is expensive and difficult. The mineral would have to be shipped at the port of Sarco.

At about 15 miles from the town of Serena, near to the Longitudinal Railway now in course of construction, and in a straight line six miles from the coast, but separated from the coast by the high mountain of "Juan Soldado," there have been discovered and even exploited, several important deposits of the "Romeral." The "Romeral" mineral appears to be rich in iron and of exceptional purity. The analyses made at the School of Mines in Paris on the various samples gave 69 to 70 per cent. of iron, no sulphur, and phosphorus not exceeding 0.006 per cent. These ores could be transported by rail to the excellent port of Coquimbo for shipment, or by aerial ropeway or double-inclined planes to a small bay situated at a distance of six to seven miles.

At some miles from Coquimbo, near the mountain of "Pan de Azucar," appear the important outcrops of Huachalalumé, which have been proved profitable up to a certain point. The port of shipment for these ores would be Coquimbo. At a short distance from the line of rail which unites Coquimbo to the town of Ovalle, there have been exploited deposits of good ore of fair thickness and extent. This has been shipped as flux to the northern works, Bella Vista and Playa Blanca, near to Antofagasta, where copper and silver ores are smelted. At a depth—and this is an exception—these ores seem to deteriorate.

At some little distance from the town of Ovalle, iron ore is to be found on the plain in the form of boulders and rubble. These deposits are spread over the hills of Dorado; they are believed to have a future before them for export to Europe and the United States. As regards transport to the coast, the ore would be conveyed by the line of rail to Tongoi, which would be the shipping port, or the line to Coquimbo could be utilized. In the province of Coquimbo, in the Department of Serena, the famous Tofo iron-ore mines are found, the most important in Chili, both as regards quantity and quality. Speaking generally, the iron ore from the mines mentioned above varies between 60 and 69 per cent. in iron, but no particulars are available as to the content of silica, phosphorus and sulphur.

The exploitation of manganese in Chili reached its highest point during the period 1884-1906. About 500,000 tons of manganese ore, assaying 50 to 52 per cent. of metallic manganese, were then extracted, having a value of £140,000, but owing to the great fall in price of manganese the working of the mines, for the time being, was stopped. The manganese deposits in Chili, at least up to a certain depth, often very limited, are of some importance. The ore, assaying 50 to 54 per cent. manganese, is found in the form of pyrolusite or braunite, with calcareous gangue, generally in lodes with a width varying between 3 ft. and 10 ft. It is particularly in the provinces of Atacama and Coquimbo that manganiferous deposits are met with, but this metal has been found, and even exploited, in almost the whole of the territory from north to south. It has even been found in the southern regions near Valdivia. There may also be mentioned, from the point of view of the future, the large deposits of manganese belonging to Mr. Enrique Valdès Gonzalez, situated in the north of the Department of Serena, about 55 miles east of Apolillado

*From a paper read before the Société des Ingeneurs Civiles de France.

bay. These deposits consist of numerous lodes, 6 ft. 6 in. to 10 ft. wide. Mining is easy, and it would be possible to ship very important quantities of manganese ore of a grade varying between 48 and 52 per cent. metallic manganese. These deposits, like many others in Chili, will only have a commercial value after they are connected to the coast either by a special railway, or by taking advantage to a certain extent of the proximity of the Longitudinal Railway now in course of construction.

Fuel.—In some districts, principally in the south, wood or charcoal can be obtained by using the timber supplied by the forests, which cover, as far as the sea, a part of the southern territory. In the central districts (Parral, etc.), where the iron-ore mines are found, it could also be procured, but in limited quantities. It is also in the south of Chili, near Concepcion, Lota, Coronel, Lebu, etc., that the Chilian lignites are found. These lignites are now being mined. They supply a fairly good fuel, used not only on board steamers and for the manufacture of gas, etc., but also in manufacturing industry for the smelting of minerals in reverberatory furnaces.

Labor.—Proof has been given in the mining and metallurgical industries established in Chili of the exceptional qualities, both as regards strength and intelligence, of the Chilian workman.

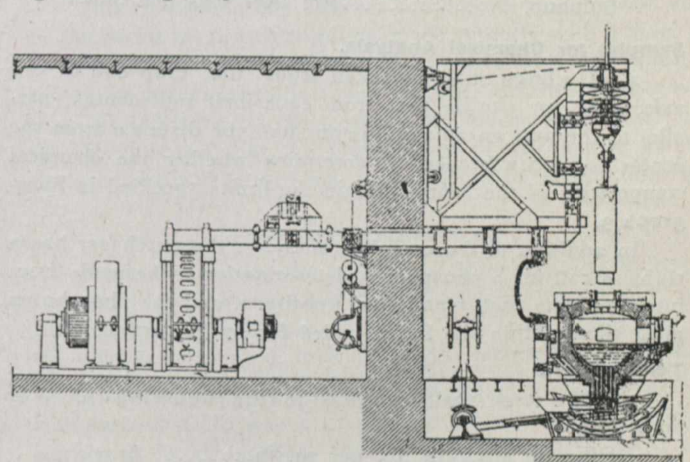
ELECTRIC FURNACE AT THE BURBACH WORKS.*

Owing to the losses and inconveniences arising from the addition of solid ferro-manganese to steel charges, many steel makers adopted the practice of melting the ferro-manganese in a crucible. This method, however, proved too costly, in consequence of the vaporization of the manganese, until the idea arose of employing the electric furnace for that purpose, and the experiments made by Keller demonstrated its applicability. A Keller furnace, a sectional elevation of which is shown in Fig. 1, has been installed at the Burbach Works, where it is used both for steel-making and for remelting ferro-manganese, over 1,000 tons having been treated, in the intervals of using the furnace for making steel, during the past nine months. When ferro-manganese is being melted, the furnace works continuously, being kept warm over Sundays with a low consumption of current. Any repairs that are found necessary can be executed after pouring the contents of the furnace into a ladle, to be returned when the job is completed. The replacement of the electrode is quickly effected, the worn-out electrode being lifted out by a swing jib, and the new one lowered into position by a second jib of similar character.

The behavior of the lining is an important factor in the cost of re-melting ferro-manganese; but in this furnace work has been carried on for three months at a time before re-lining became necessary, although the acid lining of the vault was already worn out in working previous steel charges. Even the dolomite lining only needed occasional patching. A still longer life could be ensured by using magnesite lining, which would last, as in the pig-iron mixer, from six to nine months. The sloping bottom lining only needed occasional patching, and the hearth also stood the nine months' continuous work, the bars neither melting nor being converted into ferro-manganese.

The furnace is started by simply lowering the electrode on to the solid charge of ferro-manganese, without it being first necessary to pour any pig-iron or steel into the furnace—this measure, recommended by some, being inconvenient and, to some extent, impracticable, since basic iron is unsuitable on account of its high phosphorus content, whilst steel is not often available at the time when the furnace is generally started (Sunday night). Little attention is required, one man and a boy being able to run the furnace. The most convenient place to set up the furnace is on the platform of the converter, so that the molten ferro-manganese can be easily transferred in ladles. The furnace is tilted by a hydraulic machine connected with the power plant of the converters.

The experience gained at the Burbach Works goes to show that there is no loss of manganese by vaporization in the Keller furnace. The average manganese content of ten charges of solid ferro-manganese, before melting in the furnace, was found by analysis to be 80.29 per cent. whilst the mean of 37 samples of molten ferro-manganese drawn from the furnace was 80.22 per cent. of manganese, the loss being therefore infinitesimal. Although, owing to local circumstances, the charges at the Burbach Works require to be considerably overheated, no red fumes are given off to inconvenience the furnace hands. It is found that by using the ferro-manganese in molten condition a saving of 35 per cent. is obtained by comparison with solid ferro-manganese the same quality of steel being produced. Another saving arises from the fact that the ferro-manganese dust produced in crushing the lump material, or during storage, can also be melted in the furnace with ease. With the molten ferro-manganese, too, the amount of manganese incorporated with the steel can be lessened to some extent without impairing the rolling properties of the latter.



Keller Electric Furnace at the Burbach Works.

With current at ⅛d. per kw.-hour, the cost of melting the ferro-manganese works out at 20s. per ton, including current, cost of electrodes, wages, lining and repairs. Even with the present low price of ferro-manganese (£8 per ton), each 1 lb. of ferro manganese saved in a ton of steel is equivalent to a saving of £400 per annum on an output of 100,000 tons of steel; and so on in proportion.

When molten ferro-manganese is used, the desired quality of steel can be obtained with greater certainty than by the aid of solid material. In the latter case, losses in the converter are inevitable; whereas, by adding molten ferro-manganese to the steel in the ladle, it is completely absorbed by the metal, more especially since the slag can be kept back, by stiffening it with lime, before the steel is run into the ladle. By examining a sample after blowing, a reliable estimate can soon be obtained of the amount of ferro-manganese needed to produce steel of a given quality; and this circumstance is particularly valuable when large additions of ferro-manganese are given in order to produce hard metal. On the other hand, if solid ferro-manganese be added in the converter, there is always the risk that it will not be melted in time, particularly in the case of cold charges, etc. This can-

*F. Schroeder in "Stahl und Eisen."

not occur with molten ferro-manganese, and red-shortness is therefore practically impossible. The absorption of the manganese also proceeds more uniformly, the mixing being more complete, deoxidization is more energetic, and rephosphorization is precluded. Finally, a saving of time is effected by using molten material, the output of steel being, therefore, correspondingly increased.

SPECIFICATIONS FOR HEAT-TREATED CARBON STEEL AXLES, SHAFTS, AND SIMILAR PARTS.*

Process of Manufacture.

1. Steel under this specification shall be made by the open-hearth or other approved process.

Discard.

2. A sufficient amount of discard must be made from each ingot to insure freedom from piping and undue segregation.

Chemical Composition.

3. The steel shall conform to the following limits in chemical composition:

Carbon	Not over 0.60 per cent.
Manganese	0.40 to 0.80 per cent.
Phosphorus	Not over 0.05 per cent.
Sulphur	Not over 0.05 per cent.

Samples for Chemical Analysis.

4. Drillings shall be taken from the crop end of one axle, shaft, or similar part from each melt represented, parallel to the axis on any radius one-half the distance from the center to circumference, to determine whether the chemical composition of the heat is within the limits specified in Paragraph 3.

In addition to the complete analysis, the purchaser has a right to call for a phosphorus determination to be made from turnings from each tensile test specimen, and the phosphorus must show within the limits called for by Paragraph 3.

Tensile Test.

5. The steel shall conform to the following physical properties:

Ultimate strength, lb. per sq. in.	85,000
Elastic limit, lb. per sq. in.	50,000
Elongation in 2 in., per cent	22
Reduction of area, per cent	45

The elastic limit shall be determined by extensometer. Above 40,000 lbs. per sq. in., each increment of load shall be not more than 1,000 lbs. per sq. in.

Specimen for Tensile Test.

6. The test specimen, 0.5-in. diameter and 2-in. gauge length, shall be used to determine the physical properties as specified in Paragraph 5. Test specimens shall be taken from the crop end of one axle, shaft, or similar part, from each treating-plant heat; if more than one open-hearth heat is represented in a treating-plant heat, a test shall be taken from each open-hearth heat represented. A full-size prolongation shall be left on each axle, shaft, or similar part.

Cold Bend Test.

7. A cold bend test shall be made from the crop end of one axle, shaft, or similar part, from each treating-plant heat; if more than one open-hearth heat is represented in a treating-plant heat, a test shall be taken from each open-hearth heat represented. The test shall be made with a ½-in. square specimen, not exceeding 6 ins. in length, around a flat mandrel with edges of ½-in. radius, and the specimen shall bend, without fracture, 180 deg. around the said mandrel.

Location of Specimens for Tensile Test and Cold Bend Test.

8. Specimens for tensile test and cold bend test shall be taken parallel to the axis of the axle or shaft and on any radius one-half the distance from the center to the circumference.

Re-Testing.

9. In case the physical results obtained from any lot of axles, shafts, or similar parts, do not conform to those called for by Paragraphs 5 and 7, the manufacturer shall have the privilege of re-treating such parts, from which new tests shall be taken by the purchaser, and these shall govern the acceptance or rejection of the lot.

Heat Treatment.

10. Each axle, shaft, or similar part shall be allowed to cool after forging, shall then be re-heated to the proper temperature, quenched in some medium, allowed to cool, and then re-heated to the proper temperature for annealing.

Warped Axles or Shafts.

11. Warped axles or shafts or similar parts must be straightened hot; that is, at a temperature above 900 deg. Fahr. and before offering the parts for test.

Quality.

12. All axles, shafts, and similar parts shall be free from cracks, seams, flaws, or other injurious imperfections when finished. Those which show such defects while being finished by the purchaser will be rejected and returned to the manufacturer, who must pay return freight.

Finish.

13. All axles, shafts, and similar parts must be rough-turned with an allowance of ⅛ in. on surface for finishing, except on collar, which is to be left rough forged. Turning must be done on 60-deg. centers with clearance drilled at point.

Branding.

14. The heat number shall be stamped on the rough forged collar. After rough turning, the manufacturer's name, heat number, individual axle, or shaft number, and inspector's mark shall be stamped at place indicated by the purchaser, except at any point between the rough collars.

Inspection.

15. The inspector representing the purchaser shall have free entry, at all times while his contract is being executed, to all portions of the manufacturer's shop which concerns the manufacture of material ordered. All reasonable facilities shall be afforded to the inspector by the manufacturer to satisfy him that the axles, shafts and similar parts are being furnished in accordance with the specifications. All tests and inspection shall be made at the place of manufacture prior to shipment and free of cost to the purchaser. The purchaser shall have the right to make tests to govern the acceptance or rejection in their own test room, or elsewhere, as may be decided by the purchaser, such test, however, to be made at the expense of the purchaser and to be made prior to the shipment of the material. Unless otherwise arranged, any protest based on such tests must be made within six days, to be valid. Tests and inspection shall be so conducted as not to interfere unnecessarily with the operation of the mill.

*From Report of Committee to American Electric Railway Association, Oct. 9, 1911.

THE SUPPLY OF SWEDISH IRON ORE TO WESTPHALIAN BLAST FURNACES.

In the year 1908 the output of pig-iron from 87 blast furnaces, owned by 17 companies, in Rhenish-Westphalia amount to 4,804,668 tons, and represented an average yield of 45.35 per cent. of iron from the ore smelted, the actual yield in different works ranging from 54.93 per cent. to 29.45 per cent. Of the above quantity of pig-iron, 67.65 per cent. was basic pig, 8.92 per cent. hematite, 7.10 per cent. foundry pig, 7.05 per cent. steel-making pig, 5.97 per cent. Bessemer pig, 1.23 per cent. forge pig, 1.16 per cent. spiegeleisen, 0.84 per cent. ferro-manganese, and 0.08 per cent. ferro-silicon. For the production of the hematite "steel-making" pig, Bessemer pig, spiegeleisen, ferro-manganese and ferro-silicon, which together constituted 24.02 per cent. of the output, ores low in, or free from, phosphorus (Spanish ores) were required, whereas the remaining 75.08 per cent., consisting of basic pig, foundry pig and puddling iron, are produced from phosphoric ores.

The iron ore raised in the Ruhr district does not amount to even ½ per cent. of the total ore consumed; but the Lahn, Sieg and Dill districts furnish about 10 per cent. to 11 per cent., and the German-Luxemburg minette district 21 per cent. The share of the ores won in other parts of Germany is insignificant, and diminishes year by year. The Westphalian district forms the chief consumer of Swedish iron ore, this being obtained principally from the well-known Kirunavara and Gellivara deposits. Next to Sweden, Spain supplies the largest quantity of ore to Westphalia, nearly the whole of the ore shipped from Spain to Germany being consumed in that district. From 1901 to 1908 Spain furnished between 7.56 and 21.67 per cent. of the total ore smelted.

NEWS ITEMS.

That Canadian steel concerns are being hurt by American competition is admitted. A steamer loaded with 2,000 tons of steel products is now on its way to Montreal, having come direct from the Gary plant at Indiana by the canal route. Owing to competition with the Uuited States firms the Canadian prices are away below a profitable selling basis. The importation of steel direct from Gary is entirely unprecedented. According to one authority "steel can be had to-day for almost anything one wants to bid." All of which is not exactly bullish in the various Canadian steel stocks.

In connection with their course in the metallurgy of iron and steel, the students of the third year in the Engineering Faculty of Toronto University paid a visit this week to the Lackawanna Steel Plant, of Buffalo, where a very enjoyable and instructive time was spent.

POWER FOR THE CITY OF WINNIPEG.

Within a few months the city of Winnipeg will be prepared to sell electrical energy. Its great plant at Point du Bois, costing $2,250,000, and capable of generating 100,000 horsepower, is rapidly nearing completion; and the business men of Winnipeg, as well as householders generally, are looking forward with keen anticipation to cheaper power.

The rates for power at present are as follows: 6 cents per kilowatt-hour up to 50 horsepower; 4 cents per kilowatt-hour over 50 to 100 horsepower; 3 cents per kilowatt-hour over 100 horsepower. Private lighting costs 10 cents per kilowatt; electricity for cooking costs 6 cents; gas, 1.20 per 1,000 feet.

ROLLING LOADS ON BRIDGES.*

By J. E. Greiner, Consulting Engineer.

Coincident with the introduction of a particularly heavy type of locomotive is always the question as to whether bridges are being constructed of sufficient strength to safely carry this heavy engine and its possible future development.

This same question has been cropping out time and time again during the past thirty years or more, and the answer has heretofore frequently been evidenced by the construction of somewhat stronger bridges, but in many cases to an extent merely sufficient to anticipate the increasing weight of rolling stock for a brief period.

During each successive revision of the specifications it was believed that the practical limits of locomotive weights and car capacities had been fully anticipated, but the fallacy of this belief has been demonstrated so frequently that now few engineers feel inclined to assert, with any degree of confidence, at what point or at what time this development will have reached its limit. It is apparent that we have not yet passed the period of expansion and development, and the question as to whether the structures now being built are of sufficient strength depends entirely upon future development in the type and weight of the rolling stock and the accuracy with which the designer has anticipated this development.

This discussion has a direct bearing on this question and is the result of an investigation made recently with a view of ascertaining the heaviest engines in operation, the requirements of bridge specifications and the anticipated development as indicated by the capacity of modern bridges. It is, therefore, hoped that the presentation of this matter at the present time will be of some practical use to those interested.

Heaviest Locomotives.

Since 1835, about the time the first bridge was built for carrying trains, locomotives have developed from the miniature 4-wheel grasshopper weighing less than 22,000 lbs. to the enormous 24-wheel articulated type weighing 616,000 lbs.

About 20 years ago the heaviest engine in service on the Baltimore & Ohio was a Consolidation weighing about 134,000 lbs.; at the present time this road has articulated engines weighing 463,000 lbs. Similar increases have taken place quite generally on other roads until the heaviest engines of each type have reached the weights given in Table 1.

This table also gives the weight and wheel base of double-header engines with their tenders for all types excepting the articulated, where a single engine with tender is used in comparison. Attention is called to the fact that the wheel bases of all double-header engines, excepting the electric types, are considerably larger than Cooper's E series generally used for bridge designs. The articulated types, being single, have shorter wheel bases than the double-headers of other types.

The weight per foot given in the last column of this table is the total weight of engines and tenders divided by the total wheel base, double-headers for all except the articulated types. This weight per foot does not signify anything in regard to the relative effects, on bridges, of the different types of engines, and, therefore, cannot be used in comparing these effects. It is given here merely for the purpose of illustrating this fact, which will be apparent upon comparing these weights with the relative stress effects given in Table 3.

*Abstracted from paper in Bulletin No. 139 of the American Railway Engineering Association.

The heaviest locomotives in actual service on thirty-six American railways are given in Table 2, which table also indicates contemplated increases.

The increases from the 22,000-lb. grasshopper used on the Baltimore & Ohio in 1835 to the articulated type weighing 463,000 lbs. has been rapid and remarkable and is illustrated by the following data, which shows the heaviest engines in actual service on the Baltimore & Ohio Railroad from 1835 to date:

Data Showing Engine Development on Baltimore and Ohio Railroad.

Type.	Date.	Weight.
Grasshopper	1835	22,000 lbs.
Winans' Camel, 8-wheel	1851	74,600 "
Perkins' 10-wheel	1863	90,800 "
Consolidation	1873	105,200 "
Consolidation	1881	108,600 "
Mogul	1886	113,200 "
Consolidation	1887	115,600 "
Consolidation	1888	125,000 "
Baldwin, 10-wheel	1890	133,000 "
Consolidation	1892	134,200 "
Consolidation	1894	160,800 "
Electric Motor	1895	190,000 "
Consolidation	1905	208,500 "
Pacific	1906	229,500 "
Articulated	1911	463,000 "

The above shows an increase from 133,000 lbs. in 1890 to 463,000 lbs. in 1911, which is about 248 per cent. in the past 21 years. There are much heavier engines in use on other roads.

The maximum axle load in 1835 was 5,500 lbs., while at present it has gone beyond 65,000 lbs., with limit not yet reached.

Table 1—Heaviest Locomotives of Each Type.

Type.	Engine Alone.		*Double-Header.		
	Weight, Lbs.	Wheel Base, Ft.	Weight, Lbs.	Wheel Base, Ft.	Weight, Per Ft.
Atlantic	214,800	30.79	728,400	127.76	5,700
Prairie	244,700	34.25	807,500	132.92	6,070
Consolidation	260,100	26.50	860,400	131.81	6,520
12 Wheel	262,000	27.08	817,400	130.15	6,280
Decapod	267,000	29.83	802,000	127.00	6,320
Pacific	270,000	35.20	865,400	142.48	6,070
Mikado	305,000	35.00	960,000	150.00	6,400
12 Wheel Articulated	334,500	30.66	473,800	64.56	7,340
10 Coupled	361,000	43.50	1,074,000	161.00	6,670
20 Wheel Articulated	478,000	59.80	703,600	99.70	7,060
16 Wheel Articulated	493,000	40.17	588,000	82.58	7,130
24 Wheel Articulated	616,000	65.92	841,600	105.82	7,950
12 Wheel Electric	300,400	38.50	600,800	86.50	6,950
16 Wheel Electric	320,000	44.22	640,000	102.84	6,220
†Cooper's E-50	225,000	23.00	710,000	104.00	6,830
†Cooper's E-60	270,000	23.00	852,000	104.00	8,190

*Weight and wheel base for articulated engines are given for one engine and tender.

†Cooper's E-50 and E-60 typical consolidation engines are given for comparison.

Bridge Specification Requirements.

The specification loading for bridge design as now in use by the various railroads is given in Table 3, which table also gives the impact allowances and permissible unit-stresses. The simplest manner of comparing these various specified loadings, including their different impacts and unit-stresses, is by reducing them to an equivalent loading on the basis of the American Railway Engineering Association Specifications. These specifications provide for a consolidation type of engine known as Cooper's E-40, E-50, E-60 series, depending upon whether the weight on each driving axle is forty, fifty or sixty thousand pounds. The equivalent loading given in the sixth column of Table 3, therefore, means that the specified loading, impacts and unit stresses, as adopted by the various railways, are practically equivalent in their effects on bridges to the Cooper's E series loading noted, when used in connection with the American Railway Engineering Association Specifications.

This table also shows changes under consideration by a number of railways. It will be observed by reference to the table, column 6, that eleven roads are building bridges for a strength practically equal to E-60 bridges, four for E-57, seven for E-55, one for E-53, eleven for E-50, four for loads under E-50 and one for loads over E-60. Of those roads which are now designing bridges for E-50 or under, two propose the change to E-60 and three to loading in excess of E-50 in the near future.

Table 2—Heaviest Locomotives in Actual Service on 36 American Railways.

	Locomotives in Service.		Under Consideration.	
Railway.	Type.	Weight Lbs.	Type.	Weight, Lbs.
N. Y., N. H. & H.	Pacific	229,500	Pacific	235,000
B. & M.	Pacific	equal to E-43	....	....
N. Y. C. Lines	Pacific	266,100	....	....
Erie	Consolidation	260,100	Mikado	305,000
P. R. R.	Pacific	269,800	....	....
L. V.	Pacific	241,400	....	....
P. & R.	Consolidation	222,000	....	....
B. & O.	Mallet	463,000	....	....
N. & W.	Mallet	400,000	....	....
C. & O.	Mallet	392,000	Mallet	400,000
Virginian	Mallet	455,000	....	....
S. A. L.	Consolidation	212,000	....	....
Southern	Mallet	366,000	....	....
A. C. L.	Consolidation	171,000	....	....
L. & N.	Consolidation	224,000	....	....
Wabash	Consolidation	223,800	....	....
B. & L. E.	Consolidation	254,000	....	....
I. C.	Consolidation	223,000	Mikado	280,000
Pere Marquette	Consolidation	217,000	....	....
M., St. Paul & S. S. M.	Pacific	253,800	....	....
C. & A.	Mallet	323,400	....	....
C. & N. W.	Pacific	238,000	....	....
Great Northern	Consolidation	216,600	....	....
C., M. & St. P.	Mikado	260,500	....	....
C., B. & Q.	Mallet	354,500	Mallet	463,000
A., T. & S. F.	Double Santa Fe	616,000	....	....
C., R. I. & P.	Consolidation	238,900	....	....
N. P.	Mallet	435,200	....	....
M. P.	Pacific	251,000	Mallet	?
S. P.	Mallet	437,000	....	....
St. L. & S. F.	Mallet	416,000	....	....
M., K. & T.	Pacific	228,000	....	....
Grand Trunk	Consolidation	211,200	Mikado	275,000 abt.
Canadian Pacific	Mallet	261,900	....	....
C. N.	Consolidation	181,400	Consol	?
N. Rys. of M.	Mallet	338,000	....	....

It may be reasonably assumed that the specifications in force, or the proposed changes, represent the views of the engineering department of the various railways relative to the sufficiency of the present requirements for meeting future conditions, and on this assumption:

One road considers E-65 insufficient,
Thirteen roads consider E-60 sufficient,
Fifteen roads consider E-55 sufficient,
Ten roads consider E-50 sufficient.

In order to determine the relative effects, on bridges, of the various heaviest types of engines in service and the usual specification E-50 and E-60 class, the maximum shearing and bending stresses produced by each type were calculated for spans ranging from 10 ft. to 100 ft., all locomotives, excepting the articulated types, being considered as running double-headers drawing a train of 5,000 lbs. per foot of track. On the assumption that the maximum stress produced by E-50 class is represented by unity, the proportional maximum stress produced by the various locomotives on bridges under 100 ft. is given in Table 4.

It is fortunate for our bridges that the stresses produced by the heaviest engines are not in direct proportion to the weight as compared with E-50 type. For instance, the 24-wheel articulated engine weighs 174 per cent. more than E-50, but produces increased stresses varying from 15 per cent. to 33 per cent. The 16-wheel articulated type weighs 119 per cent. more, but produces increased stresses varying from 26 per cent. to 34 per cent. The 20-wheel articulated type weighs 112 per cent. more, while the stresses are increased only from 1 per cent. to 14 per cent. The 10-coupled engine weighs 60 per cent. more, while the stresses are increased from 0.0 per cent. to 26 per cent. Other engines which weigh considerably more than the E-50 produce stresses ranging from 83 per cent. to 116 per cent. of those caused by the E-50, and the excess stresses are mostly in very short spans.

The above refers to spans under 100 ft. For greater lengths the stresses will in many cases be less, and in no case will they be in excess of those mentioned above.

Capacity of Bridges.

All bridgemen know that properly designed bridges, as well as steel hopper cars, may be loaded considerably beyond their nominal capacity, and that they will carry a definite amount of overload regularly and continuously without requiring any closer attention than usually bestowed under ordinary good maintenance conditions. The capacity for overload provides to a large extent for future increases and developments.

successful practice of a number of railway engineers. Therefore, it should be clearly understood by the operating officials of railways that a bridge of the nominal E-50 capacity, that is, one designed for Cooper's E-50 loading in accordance with the American Railway Engineering Association's Standard Specifications, will not reach its full regular traffic capacity until the different classes of engines now in service shall have about the weights given in Table 5, and an E-60 bridge not until these engines have increased to the extent shown in Table 6.

An examination of these tables will show that the regular service capacity of an E-50 or an E-60 bridge will take care of engines having an increased weight over those now in service to the following extent:

Types.	E-50.	E-60.
16 and 24-Wheel Articulated	12 per cent.	34 per cent.
10-Coupled	19 per cent.	43 per cent.

Table 3.—Bridge Specification Loading.

Railway.	Engine. Type.	Weight, 1,000 Lbs.	Impact.	Tensile Unit.	Equiv. Loading.	Proposed Changes.
P. R. R. West	excess	60.0	....	$7{,}000\left(1+\frac{M}{M}\right)$	E-65	10 per cent
N. Y., N. H. & H.	E-60	270.0	A. R. E. A.	16,000	E-60	....
A. C. L.	"	270.0	"	16,000	"	....
B. & L. E.	"	270.0	"	16,000	"	....
Pere Marquette	"	270.0	"	16,000	"	....
C., C. & O.	"	270.0	"	16,000	"	....
G. N.	"	270.0	"	16,000	"	....
C. & O.	Artic.	468.0	"	16,000	"	....
C., B. & Q.	Consol.	252.0	Special	$10{,}000\left(1+\frac{D}{D+L}\right)$	"	....
A., T. & S. F.	"	291.0	....	Special	"	....
W. Md. Ry.	Artic.	488.0	Special	16,000	"	....
P. & R.	E-55	247.5	A. R. E. A.	15,000	"	....
S. P.	Consol.	240.0	....	Special	E-57	....
N. & W.	Special	275.0	Special	15,000	"	....
Virginian	E-60	270.0	A. R. E. A.	17,000	"	....
C., M. & St. P.	E-55	247.5	....	Special	"	....
Southern	E-55	247.5	A. R. E. A.	16,000	E-55	....
I. C.	"	247.5	"	16,000	"	....
C. & N. W.	"	247.5	"	16,000	"	....
C., R. I. & P.	"	247.5	"	16,000	"	....
St. L. & S. F.	"	247.5	"	16,000	"	....
Nat. Rys. of M.	E-60	270.0	....	Special	"	....
C. & A.	E-50	225.0	....	Special	"	....
N. Y. C. Lines	E-60	270.0	A. R. E. A.	18,000	E-53	....
B. & M.	E-50	225.0	A. R. E. A.	16,000	E-50	....
Erie	"	225.0	"	16,000	"	E-60
Wabash	"	225.0	"	16,000	"	....
M. P.	"	225.0	"	16,000	"	E-55
M., K. & T.	"	225.0	"	16,000	"	....
Grand Trunk	"	225.0	"	16,000	"	....
Can. Pac.	"	225.0	"	16,000	"	....
B. & O.	"	225.0	"	16,000	"	....
M., St. P. & S. S. M.	E-55	247.5	....	Special	"	....
L. & N.	Consol.	232.0	A. R. E. A.	17,000	"	E-53
N. P.	"	233.0	....	Special	"	....
L. V.	E-50	225.0	....	Special	E-47	E-60
S. A. L.	"	225.0	A. R. E. A.	17,000	"	....
C. N.	Consol.	211.5	Special	16,000	"	....
P. R. R. East	Pacific	292.0	Special	16,000	E-45	Mallet

We know from numerous tests and long experience that bridges properly designed and constructed of proper material and with members proportioned in accordance with specifications equally as good as the standard adopted by the American Railway Engineering Association, so long as maintained in good condition, will safely withstand an overload of 50 per cent. without any traffic or speed restrictions; that such a bridge may be subjected to an occasional overload considerably in excess of 50 per cent., and this without speed restrictions; and if the speed is regulated, the bridge will stand an occasional overload of 100 per cent.

This statement is consistent with the writer's personal experience with the maintenance of structures in the past 25 years, and is somewhat more conservative than has been the

Mikado, 12 and 20-Wheel Articulated Atlantic Consolidation 12-Wheel Type	30 per cent.	56 per cent.
Pacific and Decapod..	39 per cent.	67 per cent.
Prairie	46 per cent.	75 per cent.
Electric	53 to 61 per cent.	84 to 94 per cent.

The capacity of these classes of bridges when subjected to occasional loads or to regular loads operated under restricted speed will be considerably in excess of that indicated above. For example, an E-50 bridge with an overload of 75 per cent. which, when the bridge is in good condition and up to the American Railway Engineering Association Standard in design, is perfectly safe for occasional loads or regular loads under restricted speed, will carry engines weighing in excess of the engines now in use to about the extent indicated below:

16 and 24-Wheel Articulated Engines...... 30 per cent.
10-Coupled .... 39 per cent.
Mikado, 12 and 20-Wheel Articulated, Atlantic, Consolidation and 12-Wheel Type Engines .... 52 per cent.
Pacific and Decapod .... 62 per cent.
Prairie .... 70 per cent.
Electric ....79 to 88 per cent.

It will be seen from the above that loads which strain an E-60 bridge to its regular service capacity can be operated occasionally over an E-50 bridge, and even regularly when speed is restricted.

Have Present Bridges Sufficient Strength?

In view of past experience, it is perhaps reasonable to assume that some of the heavy types indicated in Table 5 as developing the full regular service capacity of an E-50 bridge may probably be operated regularly over heavy grade divisions, but experience with the present heaviest locomotives does not indicate that still heavier types will ever be proper and economical on low-grade divisions. But suppose they should be operated regularly on all divisions, whether high or low grade, then an E-50 American Railway Engineering Association Specification bridge will have ample capacity to take care of them.

Table 4.—Relative Stresses Produced by Heaviest Locomotives—Spans 10 Ft. to 100 Ft.

Class.	Actual Weight.	Proportional Weight.	Proportional Stress.	
			From	To
E-50	225,000	1.00	1.00	1.00
Atlantic	214,800	0.96	0.83	1.15
Prairie	244,700	1.09	0.88	1.03
Consolidation	260,100	1.16	0.99	1.14
12 Wheel	262,000	1.17	1.00	1.14
Decapod	267,000	1.19	0.96	1.07
Pacific	270,000	1.20	0.93	1.08
Mikado	305,000	1.36	1.02	1.16
12 Wheel Articulated	334,500	1.49	0.98	1.15
10 Coupled	361,000	1.60	1.00	1.26
20 Wheel Articulated	478,000	2.12	1.01	1.14
16 Wheel Articulated	493,000	2.19	1.26	1.34
24 Wheel Articulated	616,000	2.74	1.15	1.33
12 Wheel Electric Motor	300,400	1.33	0.83	0.98
16 Wheel Electric Motor	320,000	1.42	0.84	0.93

It is less reasonable to assume that the still heavier types of Table 6 required for developing the full regular service capacity of an E-60 bridge will ever be operated even on high-grade divisions, unless gauge of track is increased and greater clearances made, both laterally and vertically, in tunnels and bridges and the right-of-way probably also increased, or, in other words, unless all present standards are abandoned and the railway practically reconstructed.

But suppose such types can be constructed and placed in operation without changing standard gauge and clearances, they surely would not be operated regularly on low-grade divisions, and if their regular operations should be confined to high-grade divisions, then E-50 bridges on low-grade territory would have ample capacity to enable these types being transferred to and from these high-grade territories.

It appears, therefore, that an E-50 bridge is a good and economical type and provides for increased loading above the heaviest now in service to a sufficient extent to justify the railways which consider it a proper standard on all divisions until such time as conditions require practically a complete reconstruction of the railway.

It is, of course, admitted that an E-60 bridge is heavier, stronger and stiffer than an E-50 bridge. It will stand more abuse and more neglect, but it will cost from 12 per cent. to 15 per cent. more for its construction. While a number of roads have adopted this class of bridge for all divisions and others are contemplating its adoption, the justification therefor is not apparent in many cases. The mere fact that one or two roads started a somewhat radical change by building E-60 bridges should not in itself be sufficient excuse for other roads to do likewise, thereby apparently playing the youthful game of "follow your leader."

Table 5.—Full Regular Service Traffic Capacity for E-50 Bridges Based on an Overload of 50 Per Cent.

Locomotives.	Weight.	Wheel Base.	Average Axle Load.	Percentage of Increase.†
Cooper's E-75	337,500	23.00	75,000	50.0
*Atlantic	280,000	30.79	82,400	31.0
Prairie	356,300	34.25	82,600	46.0
Consolidation	342,300	26.50	75,600	32.0
12-Wheel	344,800	27.08	73,000	32.0
Decapod	374,300	29.83	66,400	40.0
Pacific	375,000	35.20	81,700	39.0
Mikado	394,200	35.00	77,900	29.0
12-Wheel Articulated	436,200	30.66	72,600	30.0
10-Coupled	429,800	43.50	71,700	19.0
20-Wheel Articulated	629,000	59.80	70,800	32.0
16-Wheel Articulated	552,000	40.17	62,800	12.0
24-Wheel Articulated	695,000	65.92	62,000	13.0
12-Wheel Electric	460,000	38.50	78,800	53.0
16-Wheel Electric	516,000	44.22	64,500	61.0

*The Atlantic type applies to spans under 15 ft.; for greater spans the weight of this class of engine would run over 60 per cent. in excess of the heaviest type now in service.

†Percentages of increase in column 5 represent the approximate increase in weight of locomotives and driving loads in excess of the maximum weights now in actual use.

This tendency toward the adoption of E-60 loading is perhaps influenced more by precedent than by good, sound reason and judgment, and is being stimulated by the bridge companies, who profit by a greater tonnage of metal used in construction.

Table 6.—Full Regular Service Traffic Capacity for E-60 Bridges Based on an Overload of 50 Per Cent.

Locomotives.	Weight.	Wheel Base.	Average Axle Load.	Percentage of Increase.†
Cooper's E-90	405,000	23.00	90,000	50.0
*Atlantic	336,000	31.79	98,800	57.0
Prairie	427,600	34.25	99,100	75.0
Consolidation	411,000	26.50	90,700	58.0
12-Wheel	413,500	27.08	87,600	58.0
Decapod	449,400	29.83	79,500	68.0
Pacific	450,000	35.20	98,000	67.0
Mikado	473,000	35.00	93,500	55.0
12-Wheel Articulated	523,800	30.66	87,100	56.0
10-Coupled	515,800	43.50	86,000	43.0
20-Wheel Articulated	754,800	59.80	85,000	58.0
16-Wheel Articulated	662,500	40.17	75,400	34.0
24-Wheel Articulated	834,000	65.92	74,400	35.0
12-Wheel Electric	552,000	38.50	94,600	84.0
16-Wheel Electric	619,200	44.22	77,400	94.0

*The Atlantic type applies to spans under 15 ft.; for greater spans the weight of this class of engine would run over 90 per cent. in excess of the heaviest type now in service.

†Percentages of increase in column 5 represent the approximate increase in weight of locomotives and driving-axle loads in excess of the maximum weights now in actual use.

The writer hopes it will not be inferred that he condemns E-60 bridges as unreasonably heavy and extravagant and, therefore, not consistent with economical construction. They are better bridges than the E-50 class, and those who are in a position to justify them in paying more for the stronger structure, or who honestly believe this reserve strength will be required in the future, should not be classed with the extravagant, since at the most it is a case of foresight and judgment.

While E-60 bridges are stronger than those of E-50 class, it is probable that if the weights of engines ever increase to an extent sufficient to develop their capacity, many of these bridges, as now being constructed, will not have sufficient clearance to enable such excessively large locomotives to be safely operated. If, therefore, E-60 bridges are constructed, it would be well to provide a lateral clearance of at least 8 ft. from the centre of the track and an overhead clearance of not less than 25 ft. above top of rail, in which case there will be some possibility of operating over them the excessively large locomotives required to develop their strength.

Those roads which prefer stronger bridges on account of severe and heavy service on high grades could reasonably adopt the E-60 as standard for high-grade divisions and E-50 for low-grade divisions.

CONCLUSIONS.

Conclusions, as they appear to the writer, consistent with the foregoing investigation may be briefly summarized as follows:—

(1) It is reasonable to assume that rolling loads of sufficient weight to develop the full regular service capacity of an E-50 bridge, as indicated in Table 5, will probably be operated regularly over heavy-grade divisions, but it is doubtful whether such types will ever be regularly operated over low-grade divisions.

(2) It is less reasonable to assume that rolling loads of the weights necessary for developing full service capacity of an E-60 bridge, as indicated in Table 6, will ever be operated even on high-grade divisions, unless present standards of gauge, roadbed and clearances are abandoned and the road practically reconstructed.

(3) An E-50 American Railway Engineering Association Specification bridge is a good and economical type with sufficient strength to safely carry, in regular unrestricted service, the heaviest locomotives that can be safely operated without a possible complete revision of present standard clearances.

(4) An E-60 bridge is heavier, stronger and stiffer than an E-50 bridge and its construction will cost from 12 per cent. to 15 per cent. more. It will safely carry the heaviest loads that it is possible to conceive of, but if the weight of engines ever increases sufficiently to develop its capacity, bridges as now constructed will probably not give sufficient clearance to enable such enormous locomotives to be safely operated.

(5) The tendency of railways is toward the adoption of E-60 bridges, but this in many cases appears to be influenced more by precedent than by good, sound reason and judgment, and it is stimulated by those who profit thereby on account of the greater tonnage of metal used in construction.

(6) If an E-60 bridge is considered warranted by the heaviest power likely to be operated, its proper place is on high-grade divisions, and it would, therefore, be good engineering practice to construct E-50 bridges on low-grade divisions, since they will have sufficient strength to permit the occasional operation to and from high-grade territories of the heaviest equipment which could be operated on the E-60 bridge in regular service traffic.

(7) E-60 bridges would be more consistent if constructed with greater clear width and height than sanctioned by present standards, because this would provide for probable increased width and height, as well as weight, of the enormous rolling stock required to develop their capacity.

VITRIFIED BRICK PAVEMENTS.*

By Will P. Blair.

The extent of injury done to brick street pavements from contraction and expansion has been greatly magnified on the one hand, and the difficulties in preventing any injury at all have been enlarged upon to an exasperating degree. We freely grant that there are behaviors in structural materials, particularly of cement, brick and iron, that are not fully and completely understood, in spite of the research work that has been done by various investigators. But that is no argument at all against the use, within the range of what we do know and even beyond that which we know, even if we should encounter hazardous and strange phenomena. In fact, if we are to await a perfect knowledge and information concerning all utilitarian matters but little progress would be made and we would be without the enjoyment of many useful and pleasurable things in life.

The city of Cleveland has afforded a field of study during the past season much beyond that of any locality in the country. The season has been very changeable in temperature. Sometimes a variation has occurred of more than 40 degrees within a period of one week, but in the experience of many years preceding the temperature has not been subject to such great variations. While it is entirely fair to say that 75 per cent. of the streets of Cleveland are properly cement filled, provision for expansion and contraction has not been considered of very great importance, and to a very large extent has been neglected. Even with the streets built under contracts calling for expansion cushions, very few streets are found where the provision exists. In spite, however, of the almost entire lack of provision for expansion and contraction, out of more than 2,700 street intersections, but 27 ruptures occurred. Twelve of these were examined personally by the writer, and in every case no provision for expansion relief whatever was found. In the remaining, I was assured by Mr. Abbott, the engineer in charge of repairs in that city, the same lack was in evidence. Throughout the city quite a few cracks occurred away from intersections, but these are not really serious to the utility of the pavement because it very seldom occurs that rutting follows; at the worst, no greater injury follows than that which occurs to each individual brick where soft fillers are used. In no case, however, have they occurred where expansion cushions have been provided.

There occurred a rupture at the intersection of Decker avenue and East Eightieth street, neither of which had any provision at all for expansion relief along the curb or transversely. The expansion force necessarily found relief at the intersection. You may ask, why at the intersection? Simply because it was the only place that the concentration of the forces found in the four streets could concentrate. The force of expansion concentrated at the intersection square, in comparison with that directed against any other portion, was the greater and so the resistance was weak and could but yield to the strain. Any other part of either street could be approached by the force of expansion concentrating from but two directions, but in the intersection the force was fourfold. The compressive resistance was no more than in a single street, so that a rupture followed. Observing gentlemen in the neighborhood informed me that it raised five feet at the intersection. A nine-year-old boy with mental equipment for accuracy informed me that the men did not know

*Paper before the American Society of Municipal Improvements.

what they were talking about, but that he did, as he measured it with a rule, and the highest portion of the rupture stood just 3½ feet above the sand cushion. It could easily be discerned that there was a slight movement in these four streets at a distance away from the intersection of more than 100 feet. Evidences were apparent that along with this movement had been an outward force pushing against the curb, increasing as it approached the place of rupture.

Fifty-six hours after the rupture a change of more than 40 degrees in temperature had taken place. The contraction that followed drew the pavements away from the curb. Near the intersection and next to the curbs on either side, the contraction showed itself to be a full inch.

Another rupture, of which I was unable to secure photographs, I had an opportunity to observe while the expansive force was in operation. Evidences were easily noted 200 feet back in one direction from the intersection, in which were placed four manholes. The portion of the street approaching this intersection from the opposite direction had the advantage of a much more perfectly constructed pavement. It was built by a different contractor and under different specifications. The cement filler was in the joints, filling them completely from top to bottom. The mixture was uniform and the expansion cushion fairly adequate. This portion of the street, although subjected to the heat of the sun equal to the other portion, with the assistance of the four manholes, stood apparently immovable. It had no transverse expansion cushion, but relief from expansion was sufficient at the curb; at least with what was held in compression the brick retained perfect contact with the sand bed.

The street opposite, approaching the intersection in which the expansion movement was particularly noticed, was not so well constructed. The cement in the interstices was irregular; the sand cushion had not been properly compressed, so that there was an uneven flow of the sand in the interstices from the original rolling of the brick; there was no compressive relief apparent in the street at all. As the intersection was approached, it was observed that the outward force of the street was much more severe near the rupture than 100 feet away from it, but the shattering and the crushing were all confined within that portion of the street improperly constructed. No part of that portion of the street properly constructed was harmed at all.

It so happened that prior to the construction of the Indianapolis Motor Speedway a temporary brick surface was put down about 16 feet in width and 200 feet long, for the purpose of merely testing a brick surface as to its adaptability as a race course, before it was finally decided to brick the entire track. This particular portion was poorly constructed, particularly in the application of the cement filler; the interstices were neither filled completely to the bottom, nor were they full and flush with the surface. And, although we warned of disaster to come and urged that it be eliminated, for the sake of economy it afterwards became a part of the track as finished. But little of this entire track was finished with the expansion cushion—simply a case of putting off until a more convenient season. Along this temporary portion, no expansion provision had been made. On account of weakness of the cement filler, it was unable to sustain a uniform compressive strength with the rest of the pavement. A bulge occurred at this weak side. The superintendent of the speedway at once concluded that he ought to relieve the strain by taking out two courses of brick across the entire pavement. Proceeding from the point of the rupture in the better constructed portion of the pavement, he soon discovered that as he weakened the pavement there was a slight movement or creeping of the entire pavement. He then went to the opposite side of the track and began to take out two courses of brick at that side of the pavement, but there was a time when the equilibrium was past and the resisting force at the center of the pavement was too weak to withstand the expansion pressure and the force found relief in a sudden crush, frightening the workmen so much that one declined to have anything more to do with it.

We secured photographs which show clearly two things to have occurred: The weak portion received the greatest rupture from the crushing force; the pavement sheared in the center and crept on the sand cushion the full width of the brick more than the other portion. The other portion of the pavement, being uniform throughout in strength, did not show a rupture, but simply closed up the crevices from which had been extracted two courses, and stopped. It is clearly obvious that the expansive force of this portion of the pavement had found relief in compression. Yet in this stretching out process no crack occurred, so tough and strong was the pavement in its monolithic structure.

It had been decided by the owners of the speedway to change an overhead bridge to a subway passage. This change, in the interest of economy, necessitated taking up a strip of pavement the full width of the track. The superintendent, on account of his experience as described, was a little at loss to know how to proceed. The writer advised that, as he took out his first line across the track, he insert wooden blocks in a way to be drawn simultaneously and to do the work in the night time, taking advantage of the lowest possible temperature. This course was pursued, and an opening made, which was followed by extreme high temperature. The closure followed from either direction nearly the entire width of the brick and then the pavement, by contraction, receded one-half the width of its advance and again no crack occurred in this action of contraction and expansion.

From this observation and experience gained, we are confirmed in several matters, sustaining our No. 1 Directions. First, demonstrating conclusively the force at work, destructive of the courses of brick that are found on either side of every transverse expansion provision. It is simply a jamming together—a movement of the entire street in opposite direction toward a weak portion. Many examples of this character can easily be seen in this city of Grand Rapids. It confirms us in the view that no transverse cushion should be provided. Second, it fully supports our contention and insistence for a uniform mixture of sand and cement. It is easily discerned that much of the expansive force can be and is taken up in compression. If the pavement is uniform in strength much relief in compression is afforded and can be depended upon. Third, in every operation of compression from two opposite directions, a certain portion of relief at least is diverted to another direction. You may say that this last statement is not exactly borne out by the observations and experiences cited. But suppose it is not entirely proven and you do provide for a full and complete relief of all of the expansive forces in the other two directions by a cushion along the curb, then you have at least relieved the pavement of one-half its expansion force, and with this, together with what relief is found in compression, the pavement is relieved or held intact to such an extent that it is scarcely subject to any injury whatever from expanding and contracting forces.

To further assure us that no bad results will follow if proper provision for expansion is made along the curb, it is necessary to heed the importance of having the cement filler uniform in strength, so we insist that the sand cushion be compressed in order that there shall be no flow of sand into the joints, which should be occupied in full by the cement filler, thus affording a uniform strength throughout the monolithic structure, lessening the chances for rupture; for just to the extent that the filler lacks in strength and uniformity will a crack rupture or destruction of the pavement follow.

To what extent cracks appear in pavements, or if at all, due to the variations that follow a wet and dry condition we are unable to say. That a certain minimum amount of contraction and expansion parallels the condition, no one can doubt; but it is believed that the strength of a monolithic structure is such that a crack occurring from such cause rarely takes place. There is no question but that cracks frequently occur due to the expansion caused by frost, or more particularly resulting from the expansive force of the frozen ground underneath the pavement. Dry earth is in no way affected by low temperature. The action which is disastrous to all pavements alike, and all kinds of roadways, in fact, which results from low temperature only happens when moisture is present. The remedy, therefore, is found in perfect drainage.

We have, then, but two elements with which to deal in preventing cracks and ruptures in cement-filled brick pavements. First, simply a provision along the curb for an expansion cushion, which is an easy matter and only requires very simple implements to make effective the necessary provision. The trouble generally has been either a determination not to make proper provision or undertaking to do it without any implements at all, merely substituting some one makeshift or another for the purpose.

As to the cracks due to low temperature, it is simply a question of drainage. The manner and method of proper drainage are controlled entirely by the character of the soil and grades. Most soils are easily drained by tiles along either side, either within or without the curb, surplus water being taken out through "T" outlets at frequent intervals. Long drains underneath the roadbed are obviously objectionable for many reasons, though they are sometimes resorted to. A better method of drainage where the soil is such that by capillary attraction the moisture climbs to the highest point, is by alternate side drains heading slightly beyond the center line.

ALLEGED CAUSE OF AUSTIN DAM FAILURE.

The following has been given out by the State Conservation Commission, at Albany, regarding the recent failure of the dam at Austin, Pa. It is based upon a preliminary report by R. McKim, State inspector of dams.

"Mr. McKim found that in two vital points, which heretofore have escaped public notice," says the statement, "portions of the dam as actually constructed differed so widely from the original plans that from the outset it was doomed to failure. In the first place, he was surprised to find that at least one portion of the dam, which drawings published showed to be 30 feet thick at the base, was only 20 feet thick.

"He could find no trace of the existence of a cut-off wall, or 'key,' which the drawings showed extended the entire length of the dam from bank to bank of the stream. This cut-off wall was an extension down into bedrock of the upstream face of the dam. The designs showed that it was to be sunk four feet into the rock and be four feet thick in the direction of the flow of the river.

"The primary purpose of this cut-off wall was to prevent the impounded water from creeping under the dam and lifting it upward—a vital point. In addition to this function it was intended to prevent the sliding of the dam on the bedrock.

"A simple illustration of this latter purpose is that a box which lies on the floor may be easily slid along, but if a strip nailed to the bottom should be placed in a groove in the floor that obstruction would make sliding the box a much harder task.

"Only twice in its brief history was this dam filled with water, and then only for short periods. The first time the water rose to the top of the dam was on January 21, 1910. Two days later the dam slid down stream about eighteen inches, and the water was drawn off, as the newspapers stated at the time.

"The water never got so deep again until the rains of the last week of September, 1911, brought the water nearly to the crest of the dam again, and utter failure resulted. In view of the conditions noted above no other result was possible."

ESTIMATES ON STANDPIPES AND TANKS.

The Aberthaw Construction Company of Boston, who have erected standpipes at Attleboro, Mass., and Westerly, R.I., have made some interesting estimates as to the cost of certain standpipes and tanks as below:—

	Dia.	Ht.	Cost	Cost M. gals	Remarks.
Waltham	100	43	$25,786	$12.90	Roof steel trusses, concrete slab.
*Attleboro ...	50	100	36,000	24.50	Central pier.
Manchester, Mass.	60	72	36,000	23.60	
Lester	80	30	12,382	11.00	No roof.
*Westerly, R.I.	40	70	16,000	24.30	
Warren, R.I..	40	50	10,591	22.50	
Paris, Me. ..	80	14	7,150	13.60	No roof.
Gilbert & Bennett	30	70	9,000	24.30	
Gilbert & Bennett	30	30	8,000	50.00	On tower 40 ft. high.
Douglass	46	20	5,260	21.15	
Burlington, Vt.	30	30	5,050	31.40	On tower 25 ft. high.
Vineyard Haven	20	70	6,000	36.60	
Walker & Pratt	35	20	3,275	22.75	
Littleton, N.H.	27	27	2,080	18.10	
Danbury	20	30	7,250	130.00	On tower 25 ft. high.
Modfield, Mass	20	23	1,530	27.80	
Modfield, Mass	20	20	1,400	29.20	
Littleton, N.H.	17	17	1,080	38.60	
Fall River ...	16	15	850	38.60	
Madison, N.J.	25	130	16,000	33.40	Around old steel pipe 75 ft. high. Top part a separate tank.

* Constructed by Aberthaw Construction Company.

PERSONAL.

Mr. Alexander Potter has assumed the entire engineering work in connection with a five foot intake tunnel, one-half mile long, and intake tower; a six million gallon purification plant; a reinforced concrete, circular reservoir, 200 feet in diameter, 30 feet deep; new 24-inch pipe line to city from reservoir; pumping machinery; outlet sewer to Arkansas River, eight miles long, 48 inches in diameter, main stem; sewage disposal for the northern district of the City of Muskogee, Oklahoma.

Mr. R. H. Thompson, city engineer for Seattle, Washington, U.S.A., has resigned his position in that capacity to become engineer in charge of the new harbor works now assuming definite form at that port. Mr. A. H. Dimock has been appointed to the vacancy.

Mr. J. M. Wilson, city engineer of Moose Jaw, has resigned his position. It is understood, he, with his assistant, Mr. R. G. Saunders, will go into private practice possibly in Regina.

A Personal Correction.

An error occurred in our recent note regarding Mr. R. S. Lea, of Montreal. This should have read Mr. W. S. Lea, which gentleman has assumed the duties of waterworks engineer for Vancouver. Mr. R. S. Lea is consulting engineer for a main drainage scheme for Greater Vancouver, to be carried out by the Burrard Peninsula Sewerage Commission.

COMING MEETINGS.

THE ENGINEERS' CLUB OF TORONTO.—Nov. 23rd, 96 King Street West, Toronto. Paper by Mr. Isham Randolph, of Chicago. R. B. Wolsey, Secretary.

AMERICAN ASSOCIATION FOR HIGHWAY IMPROVEMENT.—Nov. 20-24. First Annual Convention. Richmond, Va. Logan Waller Page, President, United States Office of Public Roads, Washington, D.C.

THE CANADIAN SOCIETY OF CIVIL ENGINEERS.—Nov. 30th. Meeting of the Toronto Branch at the Engineers' Club of Toronto, 96 King Street West, Toronto. Paper on "The Niagara River Boulevard." E. A. James, Secretary.

THE ENGINEERS' CLUB OF TORONTO.—Dec. 13, 96 King Street West, Toronto Luncheon at 1 p.m. Address by the Hon. Mr. Justice William Renwick Riddell. R. B. Wolsey, Secretary.

THE CANADIAN PUBLIC HEALTH ASSOCIATION.—Dec. 13-15. Montreal. F. C. Douglas, M.D., D.P.H., Secretary, 51 Park Avenue, Montreal. (The date of the meeting has been changed from Nov. 21-23 to Dec. 13-15)

PROVINCE OF QUEBEC ASSOCIATION OF ARCHITECTS.—Tuesday, Dec. 19th, 1911, lecture by Dr. T. A. Starkey, of McGill University, Professor of Hygiene, on "Ventilation of Public Buildings." No. 5 Beaver Hall Square, Montreal. J. E. Ganier, Secretary.

THE CANADIAN FORESTRY ASSOCIATION.—February 6, 7 and 8, 1912. Annual Meeting, Ottawa. James Lawler, Secretary.

ENGINEERING SOCIETIES.

CANADIAN SOCIETY OF CIVIL ENGINEERS.—413 Dorchester Street West, Montreal. President, C. H. Rust; Secretary, Professor C. H. McLeod.

QUEBEC BRANCH—
Chairman, P. E. Parent; Secretary, S. S. Oliver. Meetings held twice a month at Room 40, City Hall.

TORONTO BRANCH—
96 King Street West, Toronto. Chairman, H. E. T. Haultain, Acting Secretary; E. A. James, 57 Adelaide Street East, Toronto. Meets last Thursday of the month at Engineers' Club.

MANITOBA BRANCH—
Secretary E. Brydone Jack. Meets every first and third Fridays of each month, October to April, in University of Manitoba, Winnipeg.

VANCOUVER BRANCH—
Chairman, Geo. H. Webster; Secretary, H. K. Dutcher, 319 Pender Street West, Vancouver. Meets in Engineering Department, University.

OTTAWA BRANCH—
Chairman, S. J. Chapleau, Ottawa; Secretary, H. Victor Brayley, N. T. Ry., Cory Bldg.

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ONTARIO MUNICIPAL ASSOCIATION.—President, Chas. Hopewell, Mayor, Ottawa; Secretary-Treasurer, Mr. K. W. McKay, County Clerk, St. Thomas, Ontario.

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ALBERTA ASSOCIATION OF ARCHITECTS.—President, G. M. Lang; Secretary, L. M. Gotch, Calgary, Alta.

ASSOCIATION OF SASKATCHEWAN LAND SURVEYORS.—President, J. L. R. Parsons, Regina; Secretary-Treasurer, M. B. Weeks, Regina.

ASTRONOMICAL SOCIETY OF SASKATCHEWAN.—President, N. McMurchy; Secretary, Mr. McClung, Regina.

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BUILDERS, CANADIAN NATIONAL ASSOCIATION.—President, E. T. Nesbitt; Secretary Treasurer, J. H. Lauer, Montreal, Que.

CANADIAN ASSOCIATION OF STATIONARY ENGINEERS.—President, Charles Kelly, Chatham, Ont.; Secretary, W. A. Crockett, Mount Hamilton, Ont.

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CANADIAN RAILWAY CLUB.—President, H. H. Vaughan; Secretary, James Powell, P.O. Box 7, St. Lambert, near Montreal, P.Q.

CANADIAN STREET RAILWAY ASSOCIATION.—President, D. McDonald, Manager, Montreal Street Railway; Secretary, Acton Burrows, 70 Bond Street, Toronto.

CANADIAN SOCIETY OF FOREST ENGINEERS.—President, Dr. Fernow, Toronto; Secretary, F. W. H. Jacombe, Department of the Interior, Ottawa.

CENTRAL RAILWAY AND ENGINEERING CLUB.—Toronto, President, G. Baldwin; Secretary, C. L. Worth, 409 Union Station. Meets third Tuesday each month except June, July, August.

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EDMONTON ENGINEERING SOCIETY.—President, J. Chalmers; Secretary, B. F. Mitchell, City Engineer's Office, Edmonton, Alberta.

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ENGINEERS' CLUB OF MONTREAL.—Secretary, C. M. Strange, 9 Beaver Hall Square, Montreal.

ENGINEERS' CLUB OF TORONTO.—96 King Street West. President, Killaly Gamble; Secretary, R. B. Wolsey. Meeting every Thursday evening during the fall and winter months.

INSTITUTION OF ELECTRICAL ENGINEERS.—President, Dr. G. Kapp; Secretary, P. F. Rowell, Victoria Embankment, London, W.C.; Hon. Secretary-Treasurer for Canada, Lawford Grant, Power Building, Montreal, Que.

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INTERNATIONAL ASSOCIATION FOR THE PREVENTION OF SMOKE.—Secretary, R. C. Harris, City Hall, Toronto.

MANITOBA LAND SURVEYORS.—President, George McPhillips; Secretary-Treasurer, C. G. Chataway, Winnipeg, Man.

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THE PEAT ASSOCIATION OF CANADA.—Secretary, Wm. J. W. Booth, New Drawer, 2263, Main P.O., Montreal.

PROVINCE OF QUEBEC ASSOCIATION OF ARCHITECTS.—Secretary J. E. Ganier, No. 5. Beaver Hall Square, Montreal.

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WESTERN CANADA RAILWAY CLUB.—President, Grant Hall; Secretary, W. H. Rosevear, 199 Chestnut Street, Winnipeg. Man. Second Monday, except June, July and August at Winnipeg.