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DISCUSSION, DESIGN, AND SPECIFICATIONS FOR A REINFORCED CONCRETE BRIDGE ABUTMENT.

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(To be read in the autumn of 1907.)

The past few years have seen a tremendous growth in the use of concrete, plain and reinforced. One has only to look at the remarkable growth of the Portland Cement Industry, both in the United States and in Canada, to realize this fact. Cements have, of course, been known for a very long time, but within the past twenty-five years the cost of the manufacture of Portland Cement has, through a great deal of scientific study, been very much reduced, and at the same time the quality and uniformity of the cement have been greatly improved. These facts in a large degree account for the great increase in its consumption.

With the increase in the use of Portland Cement Concrete, the study of its qualities has gone hand in hand. The use of steel to overcome the inability of concrete to resist tensile stresses was a remarkable discovery. This combination of concrete and steel is made possible by the fortunate coincidence of their coefficients of expansion and contraction. It virtually gave the world a new material, steel concrete, the possibilities of which, with further study and knowledge, will certainly be very great.

In the use of every new material mistakes are bound to occur, sometimes through lack of knowledge, sometimes through carelessness or through attempts at the impossible. These failures should not discourage the use of reinforced concrete when carefully and conservatively applied by one who has sufficient knowledge of its possibilities.

One of the greatest fields for the use of reinforced concrete has, so far, been its application to railway structures, such as retaining walls, culverts, abutments, bridges, buildings, etc. It is peculiarly adapted to these purposes for the following reasons:

1st. It is more economical than solid masonry or concrete.

2nd. It is more durable; for concrete, properly reinforced, can stand all stresses, including temperature and shrinkage stresses, without cracking; and steel, protected by concrete, is rust proof.

3rd. It is fireproof.

4th. There is practically no maintenance cost, since the concrete improves rather than deteriorates with age.

5th. It is a material in which the stresses can be accurately determined, and is in consequence of greater reliability than masonry.

6th. Its erection requires very little, if any, skilled labor, and any form of construction can be employed without shop-work, the only materials necessary being timber for forms, materials for concrete, and steel bars.

The introduction of any new material, of course, depends upon its initial cost; the more economical it is, the more general its use will become. For this reason reinforced concrete has already been used extensively by railroads in the United States, principally in the West. A knowledge of its properties is, of course, necessary, the lack of which, and a natural conservatism, makes some engineers reluctant to give it their unqualified recommendation. With the great increase in its use and the greater knowledge thus being gathered every day, it will not take very long before reinforced concrete will be everywhere recognized as a standard form of construction.

The abutment described in the following paper was designed by the writer in order to compare it with a standard abutment of plain concrete made by the National Transcontinental Railway Commission. The end in view was to show the greater economy of material effected by the use of concrete reinforced. The design is detailed in the attached drawings, and specifications covering the reinforced work have also been added.

Before proceeding with the design, it will be necessary to devote a few words to the formulæ employed and the assumptions made. The whole design really resolves itself into the solution of beams and cantilevers, thus making necessary the use of some theory of flexure for reinforced concrete beams. There are a great many of these theories differing from one another in several respects. The majority of these theories are what may be termed straight line formulæ. These are nothing more than approximations, or empirical formulæ, for they assume a constant modulus of elasticity for concrete in compression, whereas this modulus is a variable decreasing with increasing stress. In the formulæ employed in this discussion a parabola is assumed as the compression curve of concrete. There are two main groups of formulæ: those attempting to represent the condition of the beam under working conditions and working stresses, and from these assumptions arriving at the safe load that any beam can carry; and those representing the beam at its ultimate carrying capacity and hence at ultimate stresses, and from these assumptions arriving at the load which will cause any beam to fail, and then by the application of a safety factor to this load, determining the safe load to which the beam may be subjected. When straight line formulæ are used, that is, when it is assumed that the rate of strain or deformation of any fibre is directly proportional to its distance from the neutral axis, and that concrete in compression has therefore a constant modulus of elasticity, the area of compression may be represented as a triangle.

Providing the assumptions were correct, it would follow then that the condition of a beam under working conditions would be represented by substituting in a formula working stresses based on the ultimate stresses allowable in the materials used. In other words, the compression area at any working stress would be in the same proportion to the compression area at ultimate stress as the assumed working stress to the ultimate stress.

It has, however, been now established without doubt that the assumption of a uniform modulus of elasticity for concrete in compression is incorrect. The stress-strain curve cannot correctly be represented by a straight line. Some other curve must be assumed, and a parabola has been generally chosen as the closest approximation. It cannot be denied that with the use of straight line or empirical formulæ, safe designs may be made, but it must appeal to every engineer that a formula representing conditions as clearly as possible is much more desirable. When such a formula is derived, based on the assumption of a variable modulus of elasticity, the use of working stresses in connection with it must be condemned, principally, because at present there are in existence very few data on the condition of beams under ordinary working conditions. Nearly all the tests up to date have been to destruction, and from these the ultimate strength of beams is fairly well-known. Secondly, assuming a parabola or any other curve excepting a straight line as the stress-strain curve of concrete, the ratio of the area of the ultimate compression curve to the area of the compression curve for any working fibre stress cannot be the same as the ratio of ultimate stress to working stress. These ratios must vary as some function of the second or third power according to the equation of the curve assumed. The assumption of working stresses in a case like this will therefore naturally not give the required factor of safety, and in some cases not even be a possible condition; that is, the assumed stresses in the steel and concrete may never occur together. From the above it would seem to be far more consistent and conservative, until further knowledge on the subject has been gained, to base formulæ on the ultimate strength of the concrete and the elastic limit of the steel, applying the factor of safety to the loads.

Formulæ can further be divided into two groups, those basing the ultimate strength of a beam on the ultimate strength of the steel, and those basing the ultimate strength on the elastic limit of the steel. When calculations for the strength of beams were first made, it was naturally assumed that the working stress allowable in the steel was some factor of its ultimate strength. Closer inspection and study of tests made this very doubtful. It is readily seen that, when steel is strained beyond the elastic limit, the bond between concrete and steel is destroyed, due to the reduction of the cross-section of the steel. If the bond is one of adhesion only, it is unquestionably destroyed; if the bond is a mechanical one, there remains, of course, much resistance to slipping, but the beam is seriously weakened. The best description of the condition of a test beam at this point has been given by Prof. A. H. Talbot, of the University of Illinois, in his bulletin of September, 1904, discussing results of tests carried on under his supervision at the engineering station of the University. Prof. Talbot says in discussing beams reinforced with sufficient steel to take all tensile stresses: "The maximum load averaged about 6% more than the load at the yield point of the metal. It would seem then that for beams not having an excess of metal, the maximum load is nearly reached when the steel is stressed up to its yield point, and that the load at the yield point of the metal may be properly taken as the ultimate strength of the beam. It seems also true that the load which will stress the steel to its elastic limit, may be calculated by using the elastic limit of the naked steel for the tensile stress in the beam, and neglecting tension in the concrete."

What probably does occur in a beam when the elastic limit of the steel is reached is that, owing to the rapid extension of the steel, the neutral axis rises and the beam fails by compression of the extreme fibres of the concrete. For the above reasons, a formula in this discussion has been adopted which represents the ultimate strength of a beam at the point where the steel reaches its elastic limit.

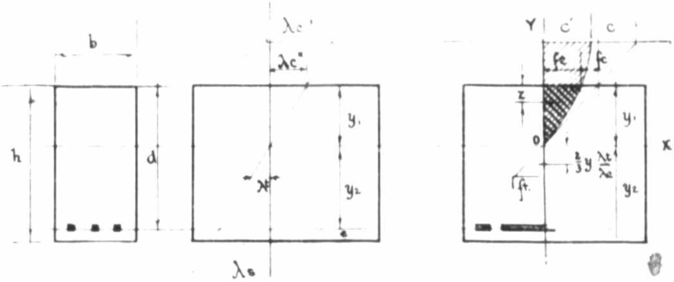
A great deal of work has been designed, using steel which has an ultimate strength of say 64,000 lbs. per square inch, and using a working stress of 16,000 lbs., the designer thinking he has a factor of safety of 4. The real factor of safety accepting the foregoing conclusions is only 2, as the elastic limit of the above steel would average 32,000 lbs. That these conclusions are correct, is pretty well conceded by all authorities in the United States at present, yet a great deal of work, designed as above stated, is still being done. This, of course, can only be to the detriment of reinforced concrete, and be the cause of unnecessary failures.

Formula.—The formula which these calculations are based upon depends on the following assumptions:

The sections plane before bending are plane after bending.

Total tension must equal total compression, thus fixing the neutral axis.

The stress strain curve is a parabola, or in other words, the compression stresses vary as the ordinates to a parabola, whose vertex is either at the top of the beam or above it. It is also assumed that the concrete is subjected to tensile stress from the



neutral axis to a point in the section where the elongation is the same as that developed by a plain beam in cross bending. This tensile value of the concrete is only assumed to make the formula as nearly correct theoretically, as possible—it has very little effect on the size of beam and amount of steel—less than 1% in fact. To obtain an equation for a parabola that would represent the variations of the modulus of elasticity of the concrete, it was considered that the modulus at rupture, was two-thirds the initial modulus, and that the resulting parabola represents closely the actual stress-strain diagram.

This formula is only applied to the ultimate strength of the beam, reinforced with what may be termed the critical percentage of steel. This percentage is such that when the steel reaches its elastic limit, the compressive stress on the extreme fibre of the concrete becomes its ultimate strength.

The compressive and tensile areas are calculated, and the compressive and tensile stresses are equated. The moment of resistance is found by taking moments about the centre of gravity of the compressive stresses, giving the following general equation:

$$M_0 = p\, d\, b\, F\,(d - z) + b\, y_1\, G \left(y_1 + \tfrac{2}{3}\, y_1 \frac{\lambda_t}{\lambda_c} - z \right)$$

Where

M_0 = resisting moment of beam (ultimate).
p = ratio of reinforcement in terms of b d.
$d = y_1 + y_2$ effective depth of beam.
b = width of beam.
F = elastic limit of steel in pounds per square inch.
Z = distance from extreme fibre in compression to centre of gravity of compression area.
y_1 = distance from neutral axis to extreme fibre in compression

$$G = \frac{E_c\, \lambda_t^2}{2\, \lambda}$$

E_c = initial modulus of elasticity of concrete in compression.
λ_t = unit elongation of concrete in tension at rupture.
λ_c = unit elongation of concrete in compression at rupture.

The above formula may seem rather complicated, but for any ultimate strength of concrete and elastic limit of steel it can be reduced to simpler ones, making the solution of beams comparatively easy.

In the following calculations it is assumed that average rock concrete is to be used. The ultimate strength of this concrete in compression is taken at 2,000 pounds per square inch in cross bending, and the initial modulus of elasticity is taken at 2,600,000. These are conservative values for a mixture of 1 part Portland cement, 3 parts sand, and 6 parts broken stone. For the steel the modulus of elasticity was assumed as 29,000,000, and the elastic limit at 55,000 pounds per square inch. From the general formula the following can be derived using these constants:

$y_1 = .3782\, d$	If $b = \frac{12}{h}$	$y_1 = .3404\, h$.		
$q = .00785\, b\, d$	and $\cdot$	$q = .08478\, h$	$= .7065$ % of cross section.	
$M_0 = .376\, b\, d^2$	10 we have	$M_0 = .2655\, h^3$	$h = 0.01654 \times M$	

where q = area of metal.

From these a table has been arranged from which the reinforcement necessary for any size of beam can be easily determined.

Steel.—It will be noticed from the above that steel with an elastic limit of 50,000 pounds per square inch has been employed in this design. It, of course, stands to reason that as the ultimate strength of the beam depends on the elastic limit of the steel, the higher this elastic limit is the more economical it will be.

To the use of high carbon or high elastic limit steel the objections may be made that it is not reliable, and that its full value may not be developed owing to insufficient adhesion.

Several years ago it was thought that economy could be effected by employing high carbon steel for bridge work. It was found, however, that owing to punching and the irregular stresses produced in plates and structural shapes, high carbon steel was unreliable. For this reason some engineers condemn its use for reinforced concrete. It should be remembered, though, that in this class of work there is no punching of the steel necessary. The stresses in the steel are nearly all tensile, and the ability of the steel to safely withstand them has been proven many times over. Shearing stresses need never be considered either, as they are always far within the shearing strength of the steel.

The steel considered in these designs is the Johnson Corrugated Bar, which has an elastic limit of 50-6,500 pounds per square inch, and an ultimate strength of 90-100,000 pounds. Large quantities of this material have been turned out, and in no case has it been found to be unreliable.

The objection that adhesion may not be sufficient to develop the full strength necessary of high elastic limit steel, is easily overcome by furnishing a suitable mechanical bond. An intimate union or bond between concrete and steel is of first importance, especially as failure of bond or lack of it may often have disastrous effects. Plain round or square bars depend on adhesion for the union of steel and concrete. This adhesion is partly due to friction, but chiefly to a mechanical bond, formed by the grout of the concrete entering into the irregularities on the surface of the bar. There are three influences affecting the adhesion and making a mechanical bond advisable: first, water percolating through the concrete (no concrete is perfectly watertight) has been proven to reduce the bond between $\frac{1}{2}$ and $\frac{2}{3}$; second, reinforcing bars when stressed, even within their elastic limit, must have their cross-section slightly reduced, and any shrinkage of the cross-section of the metal, however slight, is sufficient to materially affect the adhesion, inasmuch as the adhesion consists principally in the entering of the cement particles into the pores on the surface of the metal. If the metal has a working stress of 15,000 lbs. per square inch, then the proportionate elongation is .0005 per unit of length, with a decrease in diameter of practically one-half or .00025, by no means a negligible quantity.

Finally vibrations and shocks have also been proven to affect the adhesion. This last would alone warrant the adoption of mechanical bond reinforcement for railroad structures. To the above reasons may be added the many chances of bars being disturbed in partially set concrete during construction. From the foregoing it would seem wise to adopt a style of reinforcement with a suitable mechanical bond.

In this design, the Johnson Corrugated Bar is specified, as it has the highest efficiency of mechanical bond of any bar manufactured, its power of adhesion averaging about 2.8 times that of a plain bar of equal cross-section. It is in form of a square bar with alternate indentations so arranged that the bar is of uniform cross-section. The sides of these indentations are at an angle to the bar greater than the angle of slip of the concrete, making it necessary to shear the concrete along all sides of the bar before the bond is destroyed. As before stated, it is rolled from high carbon steel with an elastic limit between 50,000 and 65,000 pounds per square inch, and an ultimate strength of 90,000 to 100,000 pounds. It has been extensively adopted as a standard reinforcing material by railroads in the United States, such as the Chicago, Burlington and Quincy, the Wabash, the Chicago, Milwaukee and St. Paul (this road made a series of tests with all kinds of reinforcing material before adopting any particular standard; see paper by J. J. Harding, *Engineering News*, February 15, 1906), Illinois Central, Kansas City, Mexico and Orient, and many others.

General Conditions.—The general dimensions for the following design were taken from the set of Standard Abutments of the National Transcontinental Railway Commission, which were very kindly furnished the writer by Mr. H. D. Lumsden, Chief Engineer.

A rather high abutment was chosen as the economy of reinforced-concrete construction increases with the height of the structure. The distance from ground level to sub-grade was taken at 50′ 1″, the span allowed for is a 100-foot deck plate girder with the girders 9′ 0″ centre to centre, and the same width and depth of bridge seat was allowed as that shown on the standards referred to. The distance from the groundline to the bottom of the foundations was assumed as 5 feet, as shown on the standard plan, but this distance is, of course, an assumption. It depends entirely on local conditions, and would vary accordingly. The wing walls of the abutment slope back at an angle of 60° to the track. These walls are stopped at a height from which a 1½ to 1 slope will fall inside the line tangent to the face wall of the abutment at the ground line. This is the only point of difference from the standard plan. In it the wing walls are run out to a height of 4′ 0″ above the groundline. This seems hardly necessary, as by the former method a clearance equal to that between the 2 face walls is maintained, which in most cases will be all that is necessary. An allowance for this reduction in quantity has been made in the comparison of quantities and cost further on.

In the following discussion the general methods and assumptions for designing all parts of the abutment have been given. Although the whole has been worked out in detail, in order to make a proper detail drawing, where calculations of the same nature are necessary, more than once they have not been carried out in the discussion.

Foundations.—From the diagrams of sections on sheet 3 will be seen that owing to height of the abutment the resulting soil pressures are so great that in most cases, unless the foundation consisted of cemented gravel or rock, pile foundations would have to be used. Therefore, in order to make this design as modern as possible, the use of concrete piles for this purpose will first be considered. The use of concrete piles may not in all cases be economical, especially in localities where good timber piles are available and not too expensive. In a majority of cases, however their use will be found satisfactory where the following conditions obtain: where timber piles are scarce and consequently expensive, where the distance between low water line and ground level at face of abutment is considerable, and when a concrete pile is used that gives a bearing capacity much larger than that of an ordinary wooden pile. Such a pile is obtained when driven by what is called the Raymond System. It must be remembered that nearly all cases where pile-driving is necessary differ from one another, and that the exact number of piles used for any foundation is, of course, always determined on the ground, so that only a general discussion can here be introduced.

Raymond piles are of the following sizes:

20	feet	long,	20	inches	diameter	at top,	6	inches	at the point.
25	"	"	20	"	"	"	8	"	" "
30	"	"	20	"	"	"	8	"	" "
35	"	"	18	"	"	"	8	"	" "
40	"	"	18	"	"	"	8	"	" "

The method of driving and making these piles is briefly as follows:

"A collapsible steel core of a conical shape corresponding to the above dimensions is encased with a thin, tight-fitting sheet iron shell, generally No. 20 gauge. This core and its casing are driven to the required depth by an ordinary pile-driver. The core is so constructed that when the driving is completed, it is collapsed by a system of wedge surfaces, and is easily withdrawn from its casing, leaving it in the ground as a form for the concrete, and preventing the earth from closing up the hole made. The casing is then filled up with Portland Cement Concrete.

The advantages of this system of piling are:

1. The use of a shell or form for each pile.
2. The tapering shape of the pile.
3. The ease of reinforcement.
4. The fact that the concrete is not subject to blows and shocks from driving.

The shell protects the green concrete against quicksand and mud, etc., and makes it possible to ascertain that every hole, and therefore every pile, is perfect.

The tapering shape effects an economy in the number of feet of piling necessary, producing a greater bearing capacity. As it is driven into the ground, it drives harder with each blow since it has to increase the size of the hole for the entire distance of its penetration into the ground. It thus takes advantage of the full bearing power of the soil. The absence of driving on the concrete is also to be commended as, when driven, concrete piles cannot stand a hard blow of the hammer without fracture.

It is very difficult to say exactly what load these piles will safely carry, all the tests made, however, would indicate that they will bear from 2 to 3 times as much as an ordinary wooden pile.

In the *Engineering Record* of March 4th, 1905, is a paper by W. R. Harper, on tests made on some Raymond piles driven for the rebuilding of the Naval Academy of Annapolis, Md. A test pile 22½ feet long with 16 inch diameter at the top and 6 inch diameter at the bottom was driven and penetrated the distance of 1" for eight blows of a 3,000 pound steam hammer. Loaded with 41 tons it showed a settlement of 3/32", increasing to 7/64" in 10 days. Further loading to 61 tons showed a settlement of 7/16", and when the load was removed the pile rose ⅛", making a total settlement of 5/16". This rise was attributed to the elasticity of the soil. This pile was driven in soil reclaimed from the River Severn. On consideration of all the data at hand, a safe load of 30 tons per pile has been allowed in the design, the piles taking the total load.

The economy in the use of these piles lies in the fact that less lineal feet of piling are used, and there is a great saving generally, in excavation and masonry, as the tops of these piles do not have to be placed below low water line.

Design.—In general the abutment has been designed as follows: The base is about ½ the height. This dimension is greater than the 4/10 usually allowed for solid masonry abutments, and therefore makes the structure much more stable. This extension of base is also very easily obtained without much extra cost. Two main buttresses support the bridge seat and are placed directly underneath the centres of bearing of the girders. A face wall connects these buttresses to take care of horizontal earth pressures and live load pressures transmitted through the earth. The face wall supported by buttresses is continued, forming the wing walls. The buttresses and face wall rest on a continuous base which resists the earth pressures. At the back of the bridge seat is a parapet wall supported by buttresses which runs into the wing walls. The face wall is thoroughly tied to the buttresses by reinforcing bars. As the height of this abutment is considerable and the resultant horizontal thrust would therefore be large, a shelf has been placed between the four centre buttresses to reduce the overturning moment. This makes somewhat more material, but effects an economy by reducing the extreme soil-pressures and the steel necessary in the buttresses to withstand the overturning moment.

The stability of the abutment has been examined at the buttresses A, B, C, and E. The resultant pressure lines and soil pressures, etc., for these sections are shown on sheet 3 of attached drawings. The formula used for determining stresses in earthwork is Rankine's, principally because it is the safest formula giving beyond doubt pressures in excess of actual working conditions.

Let P = resultant horizontal pressure on wall in pounds.
p = pressure per square foot of horizontal pressure on wall in pounds.
W = weight of earth per cubic foot.
h = height of wall.
θ = angle of surface of backfilling.
ϕ = angle of repose of material
then at any height h:

$$P = \frac{W\,h^2}{2} \cos\theta \, \frac{\operatorname{Cos}\theta - \sqrt{\operatorname{Cos}^2\theta - \operatorname{Cos}^2\phi}}{\operatorname{Cos}\theta + \sqrt{\operatorname{Cos}^2\theta - \operatorname{Cos}^2\phi}}$$

In this discussion the worst condition was assumed, *i.e.*, that the line of cleavage of the material behind the wall is a slope of 30° to the horizontal, or that $\phi = 30°$.

At the top of the embankment θ is of course $= 0°$; this, although perhaps not quite correct, was also assumed at the other sections on account of shortening the work necessary.

The above formula then becomes—

$$P = \frac{w\,h^2}{6} \text{ and } p = \frac{w\,h}{3}$$

or the horizontal pressure per square foot at any depth on the face wall of the abutment is one-third of the vertical pressure per square foot at the same point. This assumption was also made figuring the live load pressures on the abutment.

In examining the section at buttress "A" two cases were worked out. First, the bridge was considered fully loaded with no live load on the track behind the abutment; second, the bridge was fully loaded together with a live load of 10,000 lbs. per lineal foot of track behind the abutment. The total dead load of the girder span was assumed as 200,000 pounds, and the live load of the span fully loaded as 610,000 pounds, making a load on each main buttress of 202,500 pounds. The live load of 10,000 pounds per lineal foot of track was considered distributed over 18 feet, giving a vertical intensity of 555 pounds per square foot, and a horizontal intensity of 185 pounds per square foot. The shelf between the buttresses was considered to relieve the wall from the effects of the live load below it, and to reduce the intensity of the earth pressure. The horizontal pressure below the shelf was figured as the same, due to a height of earth equal to the height of the shelf above the base. Of the 2 cases considered at buttress "A," the second gave a greater extreme soil pressure, namely, 11,590 lbs. per square foot. The resultant pressure line, however, intersects the base inside of the middle third. Referring to this, another advantage of reinforced concrete walls is obvious. As they are capable of taking tensile stresses, it is not of such importance as in plain masonry that the resultant pressure line should intersect the base within the middle third. All that has to be taken care of is that the extreme soil pressure does not exceed the safe allowable bearing capacity of the foundation soil.

Sections at buttresses B, C, and E were examined in a similar manner to the one mentioned. It was assumed that the live load would not have any effect beyond the buttresses "B" so that at "C" and "E," earth pressures were alone figured on. It will be noticed that in these other sections the resultant pressure line does not intersect the base within the middle third, but as the soil pressures do not exceed the maximum at buttress "A," it may be considered that the wall will not suffer from this. The diagrams on sheet 3 show the maximum upward soil pressures, also the downward earth pressures on the base due to the weight of the earth fill on top.

Parapet Wall.—This consists of a 16" wall supported by the continuations of two of the main buttresses. This wall is designed as a horizontal beam supported by the buttresses, which are 9' 0" centre to centre. In nearly all reinforced concrete structures beams act as continuous ones, due to the method of construction. It is, however, difficult to say exactly how much this continuity can be relied upon. Some designers neglect it altogether and figure the beam as one simply supported. It seems, however, better and more general practice to figure the moment of resistance of such a beam as $\frac{1}{10}wl^2$, reinforcing it so that the continuity can be taken full advantage of.

Size of parapet wall necessary at bottom.

Height of earth fill = to about 10′ 0″.

Horz. pressure $= \frac{10 \times 100}{3} = 333$ lbs. per square foot.

The vertical pressure due to live load at this point is assumed as 10,000 lbs. distributed over 13′ 0″ $= \frac{10000}{13} = 770$ lbs. per square foot.

Horizontal pressures are due to L.L. $= \frac{770}{3} = 257$ lbs.

To this live load pressure was added 50%, to allow for the effect of impact. Total load, therefore = 718 lbs. per square foot. In the calculations throughout a factor of safety of 4 has been allowed.

The ultimate moment on a strip of wall 1 foot wide $= \frac{1}{10} 718 \times 81 \times 12 \times 4 = 232{,}600''$ lbs. (inch pounds).

This requires a beam 8″ deep with .68 square inches of metal per square foot. The thickness of the parapet wall is 16″, but as, in order to prevent exposure of reinforcing metal, all bars are to be placed 3″ in the clear from exposed surfaces, the effective depth of the wall is say 14″. The steel can therefore be reduced. This is done proportionately to the depth, as the amount of steel necessary varies directly as the depth. Steel necessary therefore is $\frac{8}{14} \times .68 = .39$ square inch per foot.

In designing reinforced concrete, it is always important to examine shearing stresses, as they very often are the limiting factors for beams. The average shear on any section should never exceed 60 lbs. per square inch. If it does exceed this amount, shear reinforcement should be used. Shear on 12″ of parapet wall $= \frac{3.5 \times 718}{16 \times 12} = 13$ lbs.

Face Wall.—The face wall is designed by the same method as the parapet wall. It was considered better to make this wall somewhat stronger than figured, due to the fact that besides being subjected to cross bending, it is in compression due to its own dead load, and also in compression due to its T beam action in conjunction with the buttress. For this reason a batter of 1 in 24 was put on the face. Horizontal bars are hooked over the bars in the face wall and run back into the buttress. These were figured strong enough to take the horizontal reaction of the wall between buttresses, without depending on the tensile strength of the concrete. Impact loads were not considered below the parapet wall, as at that depth they will be pretty well dissipated in the embankment. The greatest stress in the face wall in the centre of the abutment will be found according to the assumption made, just above the relieving shelf. When figured as already shown, the amount of metal necessary per square foot was 0.43 square inches per square foot. ¾" bars 12" on centres are used in face and parapet walls to take care of cross-bending. This is somewhat more than actually figured; but as the increase of metal increases the cost only slightly, it is generally better to incur a small increase in cost and be on the safe side. To tie the face wall together vertical ½" bars 2' 0" on centres are used, and to take care of the reverse moment over the supports, ¾" bars 12" O.C. 5' 0" long are used.

Bridge Seat.—The main buttresses are large enough, and are designed to carry the bridge loads directly to the base, so the bridge seat has no direct load to carry. In this case it was made 2' 0" thick. It should be strong enough to tie the structure together thoroughly, and for that reason it is well to reinforce it so, that if the necessity arises, it will be capable of developing its full strength.

Reinforcement necessary for this is 22 x .085 = 1.82 square inches per square foot.

1" bars 6½" O.C. are used with transverse.

½" bars 2' 0" O.C.

The bridge seat may be figured for a possible displacement of the girder.

Dead load = 50,000 lbs. Supposing a displacement of 3' 0" were possible equal to 33000 x 36 x 4 = 4,800,000 inch pounds. This would be distributed over 2' 6" of bridge seat, and would need a depth of about 23" with about the same reinforcement as above, then shear would be $\frac{33000}{30 \times 24} = 46$ lbs. per square inch.

Base.—The base should be of sufficient width to properly distribute the load within the safe bearing values of the material. In this example, owing to the height of the abutment, the loads are so excessive, that if a rock foundation were not available, piles would be used. Assume a foundation of sufficient bearing capacity.

Average soil pressure on projection of base beyond face wall at buttress "A" = 10,300 pounds per square foot. This projection is assumed to act as a cantilever. Projection is 7' 0". In this case shear will be found to be the limiting factor.

Assuming the thickness at the face wall to be 7' 6", average shear on 12" = $\frac{10\ 3000 \times 7}{90 \times 12} = 66$ lbs.

Shear reinforcement has been used here as shown on sheet 2.

Ultimate moment on 12" = 10300 x 7 x 3.5 x 12 x 4 = 12,200,000 inch pounds.

Steel necessary to take tension $= \frac{58}{90}$ x 4.92 = 316 sq. inches per square foot or $1\frac{1}{4}$" bars $6\frac{1}{2}$" centres.

The base between buttresses behind the face wall is figured as a beam supported by the buttresses to take the upward or downward pressure, as the case may be. At "A" there is a maximum upward pressure of 5,550 lbs. behind the wall, and a maximum downward pressure of 5,800 pounds per square foot at the end of the base.

Span = 9' 0" Mu $= \frac{1}{10}$ x 5800 x 81 x 12 x 4 = 2,250,000 inch lbs.

Shear $= \frac{5800 \text{ x } 35}{36 \text{ x } 12} = 47$ pounds per square inch.

Amount of steel necessary for a 36" beam to take this load = 1.4 square inches per square foot; 1" bars 9" O.C. The top and bottom of the base are reinforced this way, the spacing of the bars in the centre being somewhat further apart, as the loads to which the base is subjected are not so great. The vertical bars in the buttresses should be carried through and hooked over longitudinal bars in the base.

The shelf between centre buttresses is figured as a horizontal beam, carrying a dead load of 3,900 pounds per square foot, and a live load of 550 pounds per square foot. The thickness of this shelf is 30", and the amount of reinforcement necessary is 1" bars 9" on centres.

Buttresses.—The main buttresses should in all cases be placed directly under the bed plates of the girders; they are widened out at the bottom to distribute their load on the base, and are reinforced by bars placed in the rear, anchored in the base, to take care of the overturning moment. In case of through spans, it will be found necessary to place an additional buttress between the two main ones. The wing walls are similarly treated, buttresses being placed 9 to 10 feet on centres sufficiently reinforced, to resist the overturning moment due to horizontal earth pressures. The buttresses were figured as cantilevers fixed at the base. In reality they will most likely act in conjunction with face as T beams, the face wall taking the compression. It will be safer, however, to figure the buttress as a simple beam. The factor of safety in figuring the ultimate moment has been reduced to 3 in this case, as this moment is greatly in excess of the moment of stability of the abutment, as a whole, and, therefore, it is not advisable to design any member to resist a very much greater overturning

moment than the stability of the abutment as a whole will allow.
Buttress "A": Depth at base = 20′ 0″.
Width = 2′ 0″.

$M_u = 325000 \times 26.7 \times 12 \times 3 = 302,390,000$ inch pounds.
or 151,200,000 " " on 12″.

Depth necessary = 202″. Area of steel $= \frac{202}{240} \times 17.17 = 14.55$ sq. in.
Therefore total area necessary = 29.0 square inches or 19 — 1¼″ bars.

These bars are extended far enough into the base to have sufficient anchorage to develop their full tensile strength. The buttresses for the wing walls are designed similarly.

In constructing an abutment on the above design, care should be taken to ensure sufficient anchorage against slipping along the foundation line by sinking the foundation into the rock if there is rock foundation, or by building a toe along the front of the wall in other material, and providing for good drainage of the foundation. If piles are used, the shear along the foundation line should be examined.

Comparison of Standard Plain Concrete Abutment and Reinforced Concrete Abutment.—Initial cost is generally the principal and most important point of comparison, and in this respect the reinforced concrete abutment makes a very creditable showing. The contents of the standard abutment from the standard plan is 1,786 cubic yards. Making an allowance of 142 cubic yards for the shortening of the wing walls as indicated, the basis of comparison would be 1,644 cubic yards. A rough estimate of the attached design places the quantities at 1,060 cubic yards of concrete, and 101,200 pounds of steel.

The contract prices per yard of concrete on the National Transcontinental Railway are $10.00 on the eastern section, and $12.00 on the western section. These figures are used. An allowance of 50 cents per yard has been made for the increased cost in placing concrete reinforced. The cost of steel, including placing, has been figured at 4 cents per pound. On the basis of $10.00 a yard, the standard abutment will then cost $16,440, and the reinforced concrete abutment $14,657, effecting a saving of $1,783. On the basis of $12.00 per yard these figures are $19,728 and $16,687 respectively, and the difference in cost is $3,041. In the first case the standard abutment is 17 per cent., and in the second 18 per cent. more expensive.

Another advantage of this abutment is its greater stability, as can be seen by comparing its section at the centre with centre section of the standard abutment as shown on sheet 3 attached. Both these sections were examined, using the same assumptions. To this may be added the saving that would be effected if pile foundations were necessary. The standard abutment having a great dead load and a much greater extreme soil pressure.

These points of advantage, together with those mentioned in the beginning of the paper on the advantages of using reinforced concrete for railway structures, should be sufficient to convince any railway engineer of the great advantages of reinforced concrete construction, and of the great possibilities of this material.

In preparing this paper references have been the catalogues of the Corrugated Steel Bar Company of Canada, Ltd., and of the Raymond Concrete Pile Company of Canada, also an article by W. W. Colpitts, Assistant Chief Engineer, K. C. M. & O. Railway, on "Structures of Steel Concrete," which appeared in the *Railway Age* from January to April, 1904. Attached to this paper are a set of tentative specifications intended to cover the carrying out of the reinforced concrete work outlined in the design.

Specifications for Concrete Work for Reinforced Concrete Abutment.

Foundations.—The foundation for the abutment shall conform to the dimensions shown on the plans. If necessary a cofferdam consisting of tongue and grooved sheeting, of at least 4 inches thickness, shall be constructed. The sides and ends of the cofferdam shall be made watertight, and during the placing of the concrete the water shall be pumped therefrom, so that the footing can be laid dry. If the bottom is so porous that it is impracticable to keep the water out, sufficient concrete shall be evenly deposited over the foundation to well calk it, after which the bottom shall be pumped out and the footing laid dry. All concrete placed under water shall consist of 1 part Portland cement, 2 parts sand, and 4 parts of broken stone, as hereinafter specified.

When the excavation has been completed, the engineer will decide whether it is necessary to use piles in the foundations or not. Where bed rock is reached, the footing must be sunk into it one foot, or as much more as the engineer may deem necessary to obtain an even and proper bearing, and a satisfactory anchorage against slipping.

If the use of piles is directed, the engineer shall determine the number and spacing of piles. The piles shall be concrete piles of the following standard sizes:

When	20	ft.	long,	diam.	of	top	shall be	20″	and	diam.	of bottom	6″
"	25	"	"	"	"	"	"	20″	"	"	"	8″
"	30	"	"	"	"	"	"	20″	"	"	"	8″
"	35	"	"	"	"	"	"	18″	"	"	"	8″
"	40	"	"	"	"	"	"	18″	"	"	"	8″

These piles shall be driven according to the Raymond system of concrete piling. The core shall be driven until it does not penetrate more than three-eighths (⅜) of an inch under the blow of a hammer weighing 2,000 pounds falling 20 feet. On withdrawal of the core, the sheet-iron casing left in the ground must retain its shape. If the casing is not to the satisfaction of the engineer, he may order another casing driven inside to make sure of a perfect pile. The concrete used in filling the shells shall be composed of 1 part Portland cement, 2 parts sand, and 4 parts broken stone. It shall be well tamped when being put into place. When the piles have been driven to the satisfaction of the engineer, any earth that has risen between the piles is to be removed, and the bed is to be rammed if so directed.

Concrete.—The concrete for the abutment shall be composed of 1 part Portland cement, 2 parts sand, and 6 parts broken stone.

Cement.—The cement used shall be some standard brand of Portland cement approved of by the engineer. All cement must conform to the standard specifications of the "Canadian Society of Civil Engineers" for Portland cement. The minimum tensile strength of briquettes, one square inch in section, from samples of the cement, shall be as follows:

Neat Cement.

24 hours in moist air	175 pounds.
7 days; 1 day in moist air, 6 days in water	550 "

One part Cement, three parts Standard Sand.

7 days; 1 day in moist air, 6 days in water	200 pounds.

Sand.—The sand used shall be a good quality of building sand, and must be clean, sharp, and angular, free from loam and other deleterious admixtures. The size of the grains should be such that not less than fifty per cent. of them shall be retained upon a sieve having holes twenty-two thousandths (0.022) of an inch square, or what is commonly called a No. 30 sieve. The size of the grains should average about the size of the mesh of a No. 20 sieve. The sand would be especially preferred if it contained a considerable amount of large particles, approaching to the size of gravel.

Broken Stone.—The broken stone shall be clean crushed limestone or trap, "*run of crusher.*" It shall be of such size that the largest piece may pass through a ring of 1½" inside diameter. The stone must be hard, sound, and of good quality, free from any condition or defect that might impair its strength.

Mixing and Placing.—All materials will be measured loose, that is, not compacted into the measuring vessel. The proportion, being specified by volume, shall be accurately obtained. The mixing will be done by a machine, and any mixing machine to be used must be approved of by the engineer before being installed.

The length of time that the material is to remain in the mixer, or the number of turns or revolutions of the mixer, must be sufficient to ensure as thorough and complete a mixture of the ingredients as shall be satisfactory to the engineer. Enough water shall be used to make the concrete of such consistency that it will pass freely into the forms and around the reinforcement; but in no case shall the concrete be of such fluidity as to permit a separation of the components through the action of gravity. Where it can be so handled a moderately dry mixture will be used, such that the concrete can be thoroughly tamped. When the concrete is too wet to tamp, or where for any reason it cannot be tamped, it must be carefully worked into all corners of the forms or moulds, and around the reinforcement in such a way as to insure that there are no voids or air bubbles, and that the concrete is as thoroughly compacted as possible. In any case the manner of adding water must be such that the quantity can be accurately controlled, and the concrete made of uniform consistency. The amount of water used shall be determined by the engineer.

After the concrete is thoroughly mixed, it shall be placed in the work within one-half hour. Concrete shall not be used after it has begun to show evidence of setting, and shall be kept entirely free from foreign matter of any kind.

When another layer of concrete is placed on one that has already set, the surface of the concrete must be cleaned of all loose material, and after wetting shall be slushed with pure cement before placing the next layer of concrete. Care must be taken to avoid getting dirt or any other foreign matter on concrete surfaces on which concrete is to be placed or which have not set. Any such concrete unavoidably mixed with dirt shall be removed and replaced to the entire satisfaction of the engineer.

No concrete shall be laid in freezing weather, unless so directed by the engineer, and any concrete that may show evidence of being damaged by low temperature shall be removed and replaced. Whenever required, the surface of concrete shall be suitably protected from cold or frost, but such protection shall not insure the acceptance of such concrete should it appear to be damaged.

Form Work.—The forms for all concrete work shall be substantial and of timber of such thickness and stiffness and so braced that they are unyielding when the concrete is placed and rammed next to them. They must be such that the finished work will accurately conform to the sizes and shapes shown on the plans, and the concrete must be so placed that the finished work will present a smooth appearance, free from all voids, lines, projections, or irregularities. The forms next to concrete surfaces shall consist of tongue and grooved stuff, planed on one side. Forms must not be removed until permission to do so is given by the engineer in charge, and in no case within 48 hours after placing the concrete.

On removal of the forms, any holes left by tie rods or by accident shall be neatly plastered to give an even finish, and all exposed surfaces shall receive one coat of a neat Portland cement wash applied with a brush.

Bridge Seat.—The top of the bridge seat shall be finished off with granitoid of the following proportions:

One part of Portland cement, two parts of sand or granite screenings, three parts of granite chips broken enough to pass a one-half (½) inch ring. The top of this granitoid is to be brought to an exact level and finished with a floated surface. Its thickness is to be not less than six (6) inches.

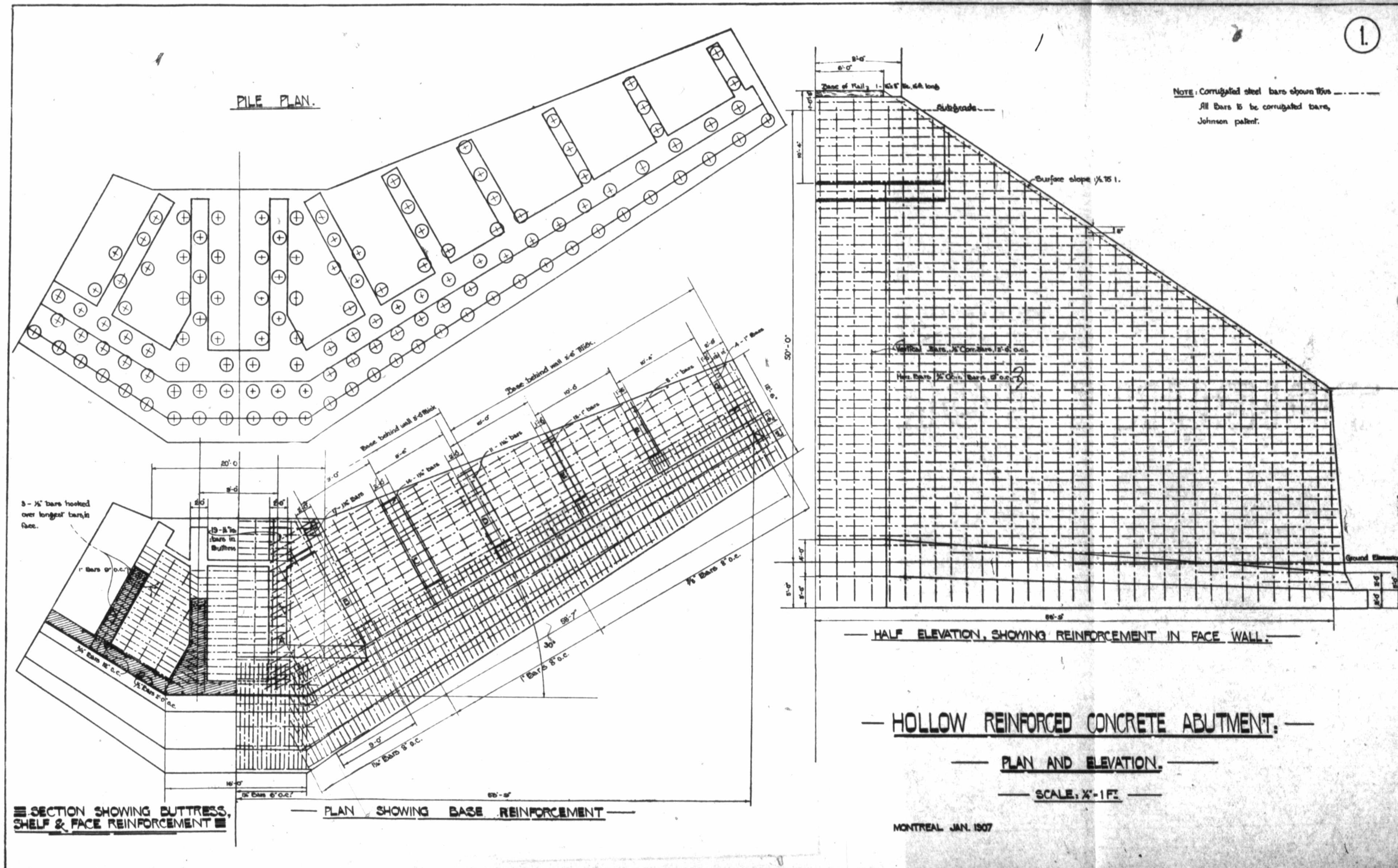
Steel Reinforcement.—All reinforcement shall consist of new section corrugated bars, Johnson patent. The size and number of these bars are shown on the drawings. They shall be made of the best grade of high elastic limit steel, of an average tensile strength of 90,000 to 100,000 pounds per square inch, and an elastic limit of not less than 50,000 pounds, and not more than 65,000 pounds per square inch. All bending of these bars, as shown on the drawings, shall be carefully and accurately done, the bars being heated in a hand forge and bent on the job.

These bars are not to be oiled or painted in any way, but are to be kept clean and as free from rust as possible. They shall be placed in proper position and maintained in this position until the laying of the concrete is completed.

All reinforcement shall be placed at best 3 inches in the clear from the surface of the concrete. When bars have to be spliced, this shall be done by lapping the bars a distance equal to 30 diameters of the bar to be spliced.

Drainage.—Provisions for draining the foundation shall be made according to instructions by the engineer-in-charge.

Inspection.—All material furnished by the contractor shall be subject to the inspection and approval of the engineer, and the engineer shall have power to condemn all work which in his opinion is not done in accordance with the contract and specifications. The decision of the engineer-in-charge will in all cases be final.

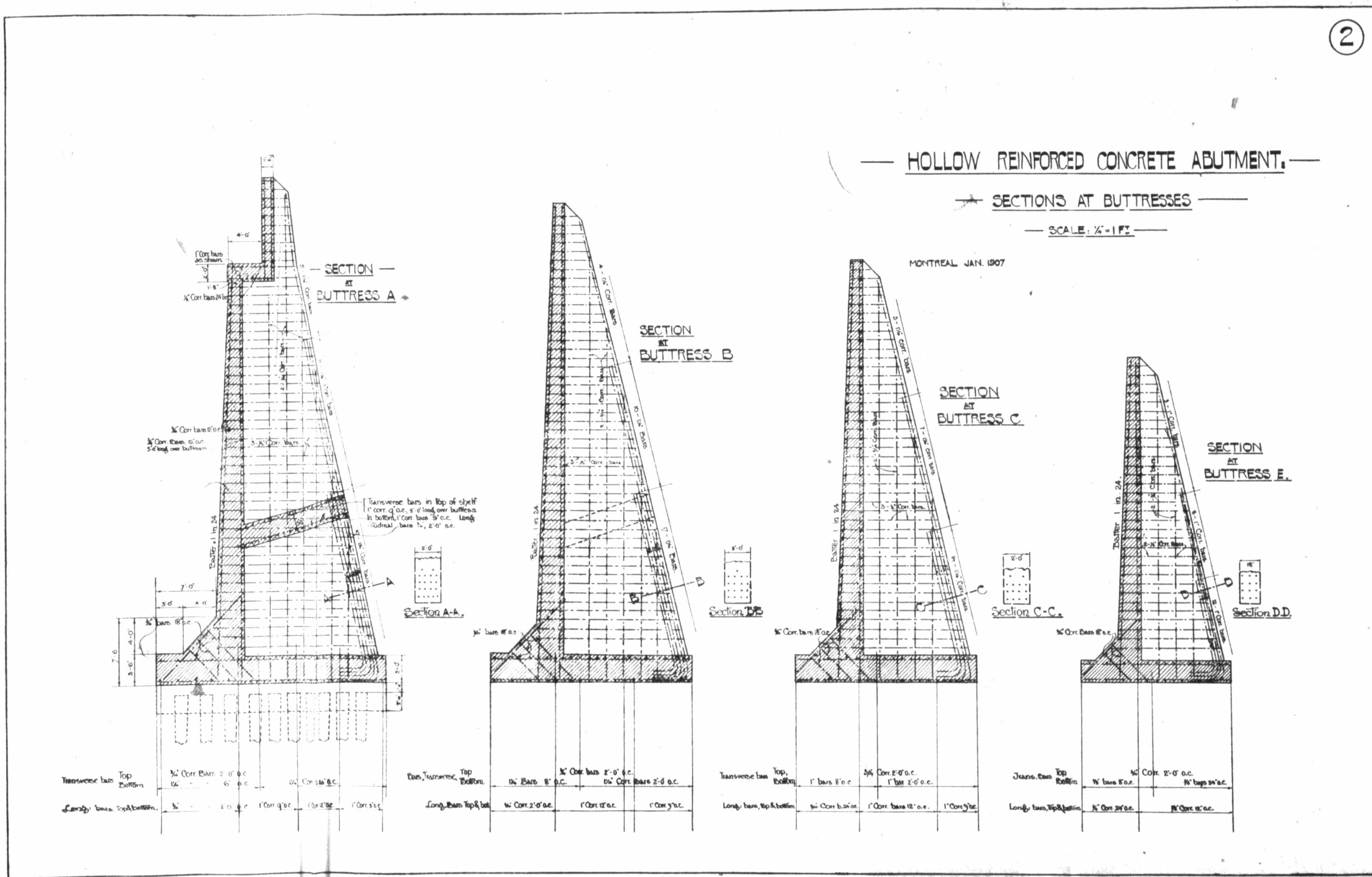


HOLLOW REINFORCED CONCRETE ABUTMENT.

PLAN AND ELEVATION.

SCALE, ¼"-1 FT.

MONTREAL JAN. 1907


2
HOLLOW REINFORCED CONCRETE ABUTMENT.
SECTIONS AT BUTTRESSES
SCALE: ¼" = 1 FT.
MONTREAL JAN. 1907
SECTION AT BUTTRESS A
SECTION AT BUTTRESS B
SECTION AT BUTTRESS C
SECTION AT BUTTRESS E.
Section A-A.
Section B-B
Section C-C.
Section D.D.
Batter 1 in 24
Transverse bars in top of shelf 1" corr. 9" o.c. 5'-0" long over buttress. In bottom, 1" Corr. bars 9" o.c. Longitudinal bars ¾", 2'-0" o.c.
Transverse bars Top Bottom
Long. bars Top & bottom
Montreal Jan. 1907

(3.)

DIAGRAMS

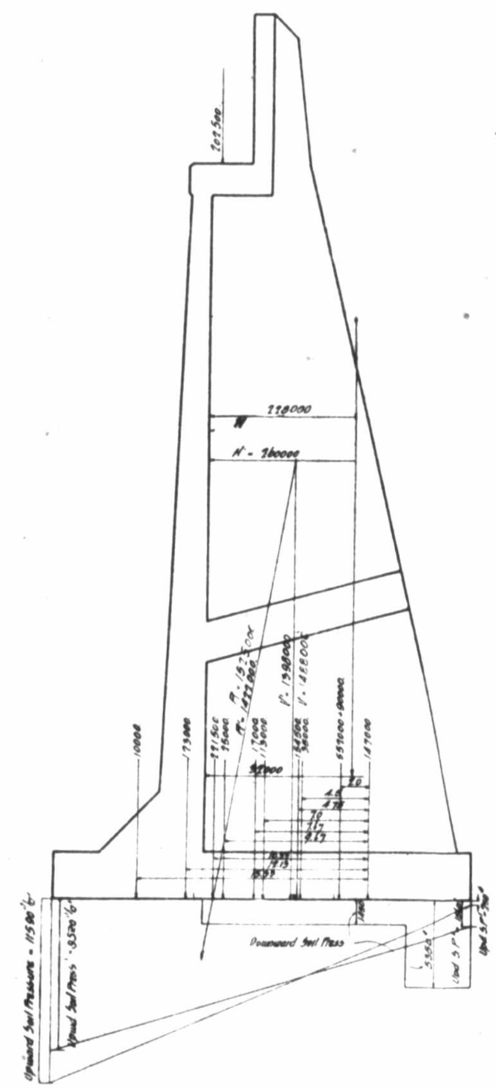
SHOWING WORST CONDITIONS OF EQUILIBRIUM & RESULTING SOIL PRESSURES AT DIFFERENT SECTIONS OF REINFORCED CONCRETE ABUTMENT

WITH SAME FOR SECTION ON ℄ OF STANDARD ABUTMENT.

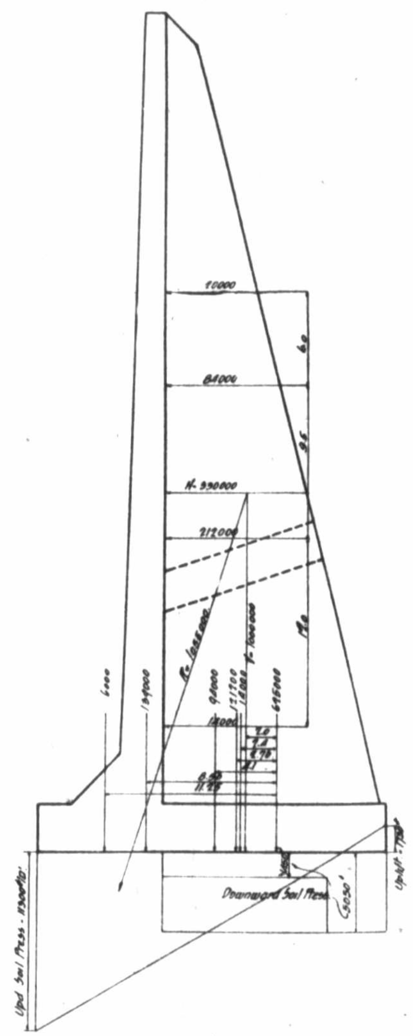
— Scale: 1/4" - 1'-0" —

Made & Trc'd by T.M. Fyshe

Montreal January 1907.



SECTION THRO' BUTTRESS 'A'



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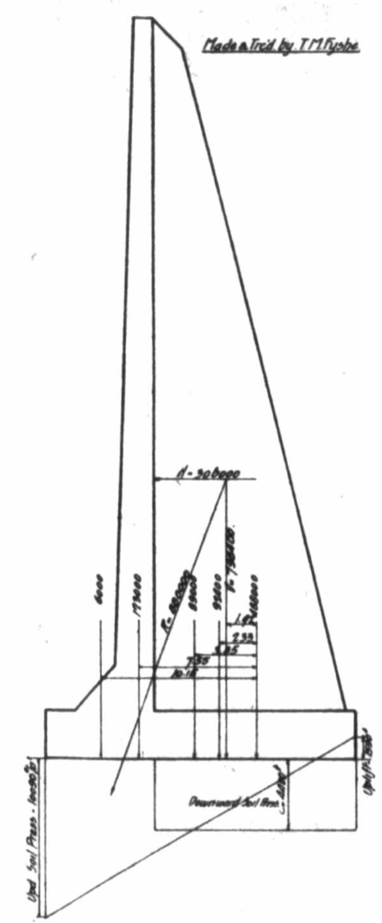
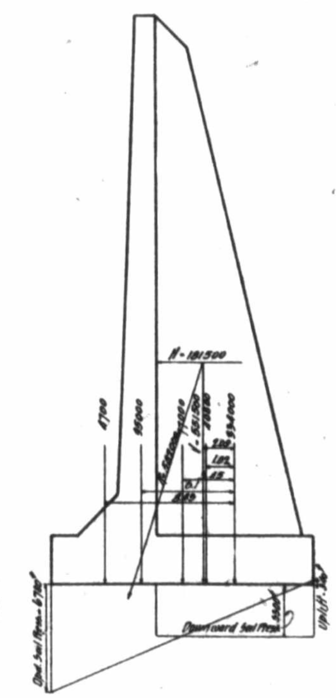
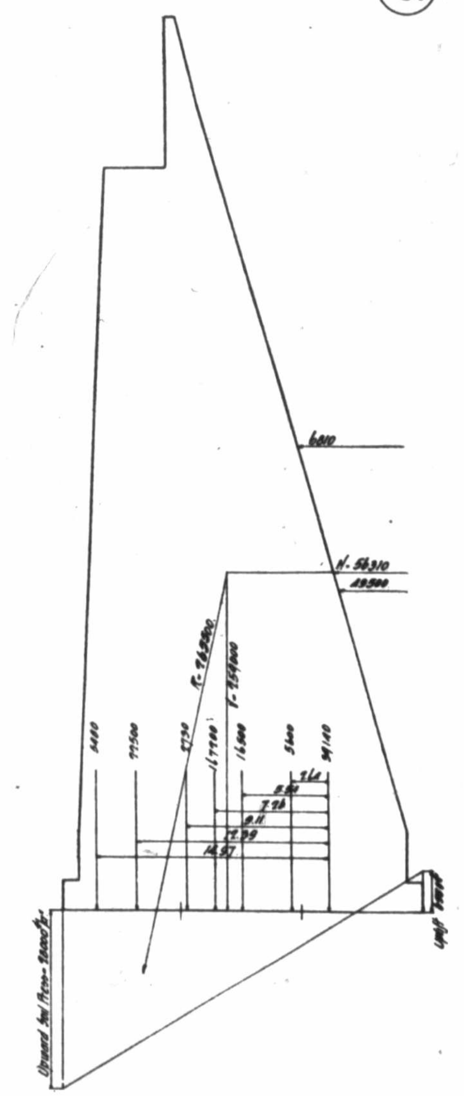


SECTION THRO' BUTTRESS 'C'.



SECTION THRO' BUTTRESS 'E'.



SECTION ON ℄ OF STANDARD ABUTMENT.

CONDITIONS FIGURED FOR LOADS ON 1'-0" OF WALL.

Note All soil pressures in pounds per square foot
Conditions figured from buttress to buttress