Vol. IX. 1895. *Part II.*

TRANSACTIONS

OF THE

Canadian Society of Civil Engineers

OCTOBER TO DECEMBER, 1895.

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The Society will not hold itself responsible for any statements or opinions which may be advanced in the following pages.

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ERRATA.

Paper 103, page 311 on 13th line from the bottom, insert in second term of latitude formula :—

$$\pm\left(\frac{m_1 - m + m_0 - m_{01}}{2}\right) R''$$

Thursday, 11th October.

THOMAS MONRO, President, in the Chair.

Paper No. 107.

SOME OPEN QUESTIONS ON THE MINOR PROBLEMS OF RAILROAD BUILDING.

By J. G. G. KERRY, A. M. CAN. SOC. C. E.

The *raison d'être* of the following paper is that the writer believes that there are many railroad engineers whose experience, like his own, has been confined almost exclusively to location and construction, and that many questions arise during construction, the correct answers to which can in reality be given only by those who have had experience in maintenance of way and operation. In the paper are embodied several questions which have arisen in the writer's own practice, and though many of them are perhaps susceptible of different answers under different conditions, he hopes in the discussion to hear opinions on them from men whose experience amply qualifies them to pronounce upon the moot points.

During construction, what is classification of material, and on what general principle is it to be studied? In many districts material may be found in all conditions from the most impalpable dust to the hardest rock, and this wide range of material must, under the specifications, be sub-divided into three or four set classifications. It may be said that the contractor in such a district bids mainly on the mercy of providence, his recollection of a past engineer and his trust in the goodwill of a coming one. This fact accounts for the strange difference in prices under similar clauses in different sections, which has been noted by all engineers whose practice has extended over a wide territory. The writer has in his note-books the records of various contracts in which the price of solid rock per cub. yd. has varied all the way from 65c to \$1.47½, loose rock from 30c to 50c, and earth from 14c to 24c. Nevertheless, it is frequently contended that material should be classified under the strict letter of the specification, and this undoubtedly would be correct wherever it is possible to write a specification which identifies every grade of material that will be met. But in districts as above described, this is impossible, and no man can for the first time go along a finished line in such a locality, and say under which of the specified grades a cut is to be placed by reason of the inherent qualities of the material, nor can he make allowance for abnormal conditions such as springs or slips which do not alter the inherent qualities of the material but do affect the cost of handling it even to 100 per cent. In such districts there seems to be three courses open: First, to let the contract with a single clause and price per cub. yd. covering all material to be handled, and making the contractor risk his bid on his own judgment of the material, which most experienced contractors are glad to do. Second, to attempt to fully cover the material with a limited number of set clauses, and to enforce the letter of the same, the phrasing of such clauses usually demanding that the benefit of all doubt shall go to the company, and the result being either abnormally high prices in the bid, or a sheriff's sale of the contractor's outfit, either of which will in the long run prove harmful to the interests of the company; or third, and most usual, to make a number of nominal classifications on which to receive bids, and to allow the resident engineers, by careful study of the conditions existing on the contract and of the handling of the material, to fix varying prices along the residency, expressing the same in percentages of the nominal grades. This does not imply that the contractor should be paid a profit on the cost of his work, but does imply that he deserves a profit on what his work ought to cost, and such a method of classification necessitates the employment of engineers of sufficient experience and reasoning power to determine whether or not the contract is being intelligently and expeditiously handled, a practice which might greatly improve the execution of other details of the construction.

Is there any real need for over-haul? The endless variations of the contract clause covering this item, the great number of enthusiasts who have thought out the one and only correct method of calculating it, and the vast file of magazine literature concerning it, all seem to indicate that this is one of the fields in which an engineer's fancy and conceit of his own notions may be allowed to run riot without material damage. Practically, the calculation of over-haul results in a great deal of tedious and burdensome work to a careful man, is usually a matter of but few dollars and cents to the contractor, and can easily be foreseen and provided for in the price per cub. yd. The only cases in which there is real necessity of an allowance for overhaul are in sections where excavations occur of great magnitude, and then a clause covering the special excavations is much simpler in practice though not on paper. As a protection to the contractor against unjust imposition, the clause is not nearly so effective as is the existence of a practical and common-sense chief engineer.

Where and when does the need of transition curves commence? Anyone who attempts to digest the mass of technical literature on this subject will be found after the effort, with a considerable library on higher mathematics, and with either a desire to regulate the speed of all the trains on the road to correspond with his pet transition, or a decided churlishness whenever the subject is mentioned. There are only two inspectors whose opinion is of material value on this point,—the rolling stock and the great unthinking public. The great unthinking public-when it is jolted from side to side in its seat, realizes only that some thing about the line is not so satisfactory as it might be, and without meditating very long on the subject it simply travels thereafter by another line if it can. The writer can recall seeing some very "sea-sick" first-class passengers on a train which was rushing through a series of very sharp curves on a trunk line without any easement near the tangent points. The locomotive is, of course, not proficient in the development of spirals nor perfectly acquainted with the intricacies of the cubic parabola, and it will often follow quite cheerfully curves whose meanderings are traceable by no known law, and in fact the precise theory of the action of the pilot is a matter of some uncertainty; but the machine does know when it is continuously exposed to unnecessary jolts and shocks, and will emphasize the fact in no uncertain figures in the auditor's records. But, perforce, neither of these inspectors can render definite answer to the definite query, and therefore an answer has to be requested of those gentlemen whose official position has afforded them opportunity to observe the opinion of the two inspectors. Such an answer should specify the degree of curvature at which the need of transition is practically felt, and to what extent the fixing of this point is influenced by varying speeds, while the particular transition used would not seem to be of material importance. The writer has heard but two objections urged to the general adoption of transition curves on new work: first, that they are unnecessary, and give a considerable amount of trouble by reason of their intricacy, a reason that can only be supported by personal laziness and ignorance, or by lack of experience; and, second, that it is impossible to file a correct and comprehensible description of the right of way in the deed books, and at the same time to have the centre line of the track as centre line of the right of way, this being the reason why one chief engineer of wide experience and unusual ability refuses to use the method.

In the operation of a built line, is there any determinable advantage in long curves and long tangents in contrast with a line of almost incessant light curvature? There are some questions in location the answers to which seem so apparent that it were folly to ask them, and yet the wrong answers are practically given again and again. Nearly every engineer can point to instances where traffic has been recklessly thrown away to save alignment, sometimes even on colossal scale, and with regard to the above minor question the writer has seen a line laid down by the locating engineers of a great system governed almost solely by consideration of long tangents and long uncompounded curves. This line proved almost ruinous to build, and its great cost could have been materially reduced by closely consulting the local topography and using the same degree of curvature more freely and in shorter lengths. The writer can recall several instances where the same love of a paper line has thrown upon the construction utterly impracticable cuts and fills, and remembers hearing a chief engineer, during the re-location of a line around an impracticable cut, urge the adoption of a line requiring an impracticable fill in order to avoid a "broken-back" of two 3° curves and a 300 foot tangent; an impracticable cut or fill being that by reason of probable slips or subsidences is likely to necessitate the handling of a much larger quantity of material than the cross sections show.

What is the cause of narrow banks on old railways? Is it because they were originally built scant in section, or because during construction they were originally built narrow and then widened by side dumping, and that as the core of the bank had been hardened by exposure, the sides had never become incorporated with it, but had gradually worked down the slope, leaving the bank with narrow top and bulged toes. The writer has seen several cases of the latter action, and it occasionally has disastrous consequences, especially when the core of the bank has been frozen and the dumped material rests against the head wall of a structure not designed to carry such pressure. The remedy, and some chief engineers insist on it, is to compel the contractor to build his banks full width from the very start. It would be safe to say that there is not much material wasted on over-wide banks.

Is there any gain in dressing the completed road-bed to curve elevation? Such dressing is undoubtedly expensive to the contractor, and calls for a considerable amount of work on the part of the engineering staff, and the result in the majority of cases is rather more of a tribute to the artistic instincts of the engineer than to his common sense. As a rule, a few weeks of wear and tear and shrinkage obliterate nearly all trace of the dressing, and the writer can recall but one case in which the practice proved of material value. This was the end section of a line being laid with rock ballast. Owing to the nature of the ground, there was an unusually firm subgrade, and as curvature was plentiful the bed was dressed to elevation, and by reason of such dressing the track superintendent was enabled to make his ballast supply, which had run short, complete the work allotted to it. Had the work been liable to much settlement, or had gravel and sand ballast been used, it is doubtful whether the saving would have been of value.

How should a line be ditched? At the time of turning over the line to the maintenance of way department there are liable to be as many varieties of ditching as there are resident engineers along the line, and these systems will vary all the way from none at all to an elaborate string of cut and berm ditches protecting every inch of the line. This is surely a matter of sufficient import to call for some settled policy, and definite instructions should be issued for the construction of a complete system of ditches while the road-bed is being built, or for the entire omission of any ditches that are not absolutely and immediately necessary. In the writer's judgment the former is the better method, for during construction there is available a force of men whose specialty is the movement of material, and they are under the eye of an engineer who has a comparatively short length of line to study, and can consequently give the matter closer attention than can the maintenance engineer of a long division, who further has to depend upon section gangs for his labour; and it must be remembered that these latter are always reduced to the scantiest number that will suffice to keep up the line, and that roadmasters and section bosses constantly claim that their forces are not enough for the satisfactory maintenance of the track, and therefore cannot do ditching. Any general orders on this subject must of course take the local conditions closely into account for the advisability of ditching varies very greatly in different sections, as an instance of which the writer has worked in sections where catch-water ditches above the slopes of a sidelong cutting were certain to cause a slip into the cut from the line of the ditch, and again in other sections where the soil was practically impermeable, and the heaviest ordinary ditching has failed to prevent the flooding of the cuts it was dug to protect.

Of ordinary railroad structures the wooden trestle is possibly the simplest, most common, and the one whose design and construction fall most frequently to the lot of the engineer. There are, therefore, almost innumerable points of dispute as to the best practice; some of which are the general type, the method of building the main posts and stringers, and the fastening of the same, the width of the floor and its fastenings, the use of rerailing guards, and the method of connecting the trestle to the earthwork on either side. From the doctrine of survival of the fittest, it would appear that the four post bent was best suited to general practice, as it is almost universally prevalent; yet there was, at least until lately, one great railroad system that used two post bents as its standard; and the writer has heard a maintenance engineer of considerable experience urge the adoption of a type having a centre post, claiming as a result of his experience that such a type would prove much easier to keep up. It has never fallen to the writer's lot to deal with any other type than the four post, and the six post as an extension of it; but as the amount of trestling on our various lines which are now in operation is large, he hopes to hear the above statement fully commented upon. The choice of the system of building up the main members is decided by a variety of circumstances; the whim of the chief engineer, the locality of the structure, the sizes and variety of the timber growing in the neighbourhood, and the required life of the structure all exercising a traceable influence on the selection of the system. The solid post type where suitable timber is in abundance is the most cheaply purchased, the most easily framed, and the most quickly and therefore most cheaply erected; but it is open to the objections that in large structures the sizes required are difficult to obtain, that the large pieces season slowly, and therefore will rapidly decay, and that the fastenings most used are very unsatisfactory from a maintenance point of view. These fastenings are generally made either by mortice and tenon, or by drift bolts driven through cap or sill into the post, and in either case it is impossible to replace one member without deranging the entire bent. Mortice and tenon work is expensive to frame, makes a place for the lodgment of the water and the commencement of decay, and cannot be repaired without considerable difficulty. Drift-bolted work is framed and put together very easily, and is very satisfactory as long as the work is new and the timber perfectly sound, which it usually is as long as the construction department has anything to do with it; but it is put together once and for all, and when any member has to be replaced it has practically to be torn to pieces before the hold of the drift-bolts can be loosened. There is used in some localities a form of fastening which is without the disadvantages of the two more common forms, and gives very satisfactory service ; it consists of dapping the cap and sill to receive the full section of the post, and fastening them to the post with a flat iron strap or stirrups, bolted through the post and passing around the cap or sill ; the bolt can be drawn at any time, the stirrup shifted out of the way, and the decayed member removed and replaced without interference of any kind with the other main members of the bent.

The full cluster type of trestle is of especial value in localities where timber of large size is scarce and the means of transportation poor. The small scantling required for the main members can generally be obtained from local timber, and although the cost of framing and erecting a cluster bent is greater than that of a solid bent when the necessary timber can be obtained on the ground, the difference is not sufficient to cover any large transportation charges. The heavier cost of the full cluster is due to the greater amount of sawing and framing per M. B. M., and to the fact that the bent must be built up piecemeal in spite of the lightness of its individual pieces. It requires a good deal more attention during construction than the solid type, and if built of timber that is liable to warp, the contractor will often have considerable trouble with the twisting of the sticks between the times of sawing, framing and erecting. It has, however, the merits, that owing to the small scantling of the timbers it seasons readily and thoroughly, thus materially retarding the date of repairs, and that when repairs finally become necessary they can be easily and promptly executed. The half cluster type is claimed by its advocates to possess the advantages of both the cluster and the solid types, without the disadvantages of either, a claim which is rather far fetched, as it in reality occupies about a mean position between the other two. In the writer's judgment, the cluster or half cluster type carefully bolted up is the most satisfactory type for permanent work, especially if the structure is of large size, and the solid trestle for all other work ; but the selection must always be modified by the size and variety of timber obtainable.

The stringers are the most important members of any trestle, and present practice has developed two distinct types of these : those of one span and those of two span lengths. The practical advantages of the one span stringers are the ease of obtaining and handling them in the small size required, and the facility with which they can be removed and replaced if properly fastened to the bent. These considerations are strong enough to justify the use of the short stringers in all but exceptional cases. Instances are known where with two rows of double length stringer laid side by side, breaking joint, and securely bolted and packed together, the stringers have carried a train across a double span caused by the destruction of a bent, and therefore it is wise in places where there is danger of washouts or other sudden removals of the bents, to use the double span stringers, in spite of the difficulty of obtaining such large sticks, the trouble of handling them, and the great inconvenience of replacing them when necessary. The necessity of the fastenings employed in general practice, to fasten the stringers to the bents, is the cause of considerable discussion, and is one of the points where some knowledge of maintenance problems would be of advantage to construction engineers. The forces tending to move the stringer are not great, and the friction between stringer and cap, when under a loaded train, is ample to resist the oscillatory and centrifugal forces. The usual practice is to bolt or drift bolt the stringers to the cap, and as it is almost impossible to draw drift bolts, and the long, thin stringer bolts are scarcely better, especially when the lumber used happens to contain acids that aid in the corrosion of the iron, it becomes necessary frequently to destroy the stringer when it is desired to shift or remove it, and as mentioned above there is ordinarily no need for such fastenings. It is not the province of the stringer to serve as a longitudinal brace to the trestle, although it undoubtedly does so; and to meet the endlong tendency developed on sharp grades, or caused by the shock of a derailed truck, a liberal dap would prove much more effectual than a single iron bolt of small section, and the depth of the stringer can be sacrificed at the end without detriment to its utility, the merits of the shoulder of the dap being greater than the possible damage caused by the splitting of the stick from the corner, which some engineers claim is the invariable result of dapping. A block spiked to the outside of each cap with a horizontal rod connecting all the stringers has proved sufficient to keep all the pieces in place, and these fastenings can be readily and quickly removed if desired.

Practice varies considerably with regard to the flooring of trestles, the lengths of the ties varying in different sections from 8 to 14 feet, the governing consideration being in most cases a reduction of first cost. There is no doubt that derailed trucks are at times carried across wide trestles with the wheels between the inner and the outer guard, and depending on the jack stringer for support; and it is advisable that the floor should be nearly as wide as the banks, but the extra width increases the cost of all the members of the trestles except the posts and stringers; and as reducing the B. M. in the trestles is one of the simplest ways of slightly lowering an apparently excessive estimate, narrow trestles have become the almost universal usage in the Middle and Southern States. In all sections the necessity of securing the ties firmly against bunching under a derailed wheel is fully recognized, and is met in the same way, *i.e.*, by a well dapped guard rail bolted at intervals to the ties and by spiking some proportion of the ties to the stringer. The custom of putting in spacing blocks between the ties is rare; but with spacing blocks between the ties and an occasional tie turned on its edge and dropped down 2 inches on to the stringer, the spiking to the stringers can be reduced to a minimum, and the floor be left in such a shape that it can be readily shifted to line at any time.

The use of guard rails on all structures and on high banks, with or without rerailing guards, is now universally accepted as good practice. With regard to the latter, the writer has heard the chief engineer of a large system assert that the rerailing blocks were a positive source of danger, and that several derailments had been caused by them on the system with which he was connected. The writer has heretofore failed to find any public record of such accidents, and would be pleased to be advised where such can be found.

What is the best method of connecting a bank with a trestle? This in particular is one of the problems which the construction engineer has to answer, and then leave the truth or otherwise of his answer to be determined by the maintenance staff. The systems of connections devised are very varied; piles have been driven before the banks were built, and the pressure of the green banks has come against the piles and pushed them completely out of place; trestle bents have been erected before the banks were built, and heavy diagonal bracing between the caps and sills of adjoining banks inserted, and if the pressure has been allowed to come against the bents, the bracing has frequently failed; and even when extra is paid for pulling the material past the bents, so that it will not press against them, it is very rarely satisfactorily done. Banks have been built and while yet green the trestle bents have been erected on mud sills laid on rough benches cut on the front slope of the banks, these mud sills varying from a half dozen short mudsills to a perfect floor of them underlaid by longitudinal pieces, and the floods have come and washed the material from under the mud sills, and the banks have settled downwards and forwards, and the trestle has been thrown completely out of line and grade. The same mud-sill method has been tried with much greater success on banks carefully dug down into wide and freely sloped benches, as this benching materially checks both settling and movements. Piles have been driven into the end of the finished banks and capped for the trestle, and this method is usually satisfactory except where the endlong slope of the old ground is sharp, giving rise to a decided downward movement of the bank and the underlying rock close so that the piles have no hold on the old ground. It is not difficult to handle a light pile driver on a $1\frac{1}{2}$ to 1 slope, and the connection is decidedly preferable to that made by a stringer sill resting on a few mud sills laid on the often scant end of a bank. It occasionally happens, especially on a winter built road-bed, that rigidly built connections prove disadvantageous for a while by reason of the sinking away of the banks and the sharp shock thereby given to passing trains; but even in this contingency, the high structure is better than the almost unsupported stringer sill. It should be needless to emphasize the fact that banks should always be built to full section, at least one-half span length beyond where the end of the trestle is designed to come. The writer has had charge of the building of connections on all of the above plans; but as said before, their real relative value can only be determined by long observation, and he would be glad of some information on this subject from those on whom the task of maintaining similar structures has fallen.

What is the best method of determining the size of small structures? The sizes are very frequently chosen by guess, and this method with careful resident engineers will generally result in structures considerably too large and therefore of needless cost. It is frequently urged that the immunity of a line from washouts is proof positive of excessive first costs. The size required is primarily a matter of locality depending largely upon the maximum rainfall and the impermeability of the soil. The best method probably to determine the capacity required is to journey along the line during the heaviest rain storm that comes, and note the flow. The writer remembers having seen an engineer double the capacity of all his culverts after one such trip, that trip being his first chance to learn how rain fell and water flowed in that part of the country. The drainage area of every hollow crossing the line should also be walked around and its area and peculiarities noted, and then a very fair estimate of the probable flow can be made; the drainage area of hollows in rough country is often much greater than can be judged from the appearance at the crossing.

What is the least depth of material to be allowed upon the covers of a box culvert? It is frequently necessary to cross fairly heavy drainage through light banks, and open structures are universally admitted to be dangerous and expensive to maintain. It is claimed that if the ties be too closely approached to the stone covers of a box culvert, the covers will be broken by the engine hammer, and it is usual in heavy freezing weather to find a hump in the track over every lightly covered box culvert; but there seems to be no generally accepted limit as to the least depth to be permitted, and statements of engineers vary very much on this point. The writer has seen an open culvert replaced by a roof of old rails carrying bricks on their flanges, and this had only about twelve inches of covering, and yet the side walls showed no effect of hammer.

There are two methods of laying the foundations of box culverts, and both have strong advocates. The first is to carry down the side walls to a safe bearing, and lay paving between them at the level at which the water is to be carried, and this is the more expensive method; the other is to pave right across the excavation, and build the side walls upon the paving. It is claimed that in the latter method the structure is liable to be washed out, because the water is practically carried below the walls, and the writer has seen such disasters take place; but its advocates say that in the first design there is nothing to hold the paving in place, and that it therefore is more liable to washing out, and the writer has seen a stream running under the side walls of a culvert built on this design; but of all the engineers he has heard discuss this question, not one has been able to give other than theoretical reasons for his preference.

Is it necessary to lay box culverts in cement mortar? The great lines make this an almost invariable practice, and yet the writer has seen many dry culverts that have given very good service, and believes that the masonry in dry culverts is better than in those laid in cement, because there is no opportunity to plaster up defective work, and it certainly costs less per yard. The dry culverts the writer has seen in satisfactory service were in a more southerly climate than the Canadian, but in one probably more liable to extreme and rapid variations of frost and thaw.

What variety of cement should be used in culvert work? There is at present a growing tendency to specify Portland cement for everything, but the wisdom of this tendency is questionable. Ordinary culvert masonry ought to be of a quality that will stand perfectly well without mortar. The duty of the mortar is to protect the masonry at its joints from destructive action, and this should be done as cheaply as possible. Everybody will concede that Portland cement bulk for bulk is greatly superior to any natural variety, but can such a statement be made cost for cost?

It is claimed that natural cements are of uncertain quality, and that they will not stand freezing before setting. If the cement be purchased from makers without reputation it may prove uncertain, but many of the large natural cement firms supply a perfectly reliable article, a fact which is evidenced by the free and successful use of it in the States, and though its failure to set after freezing prohibits its use in Canada during winter, yet that is no reason for excluding it from the structures built in summer, and these are greatly in the majority. As said before, the definite duty of cement mortar in culverts is to protect the joints from atmospheric forces, and it is not yet proven practically that this is not much better done by the free use of a cheap cement rather than by the chary use of a dear one. A series of experiments on the resisting powers and impermeability of different brands of cement cost for cost would be very useful in this connection.

Is it a fact that the tails of T abutments hammer to pieces under traffic, and if so, could they not be protected by the use of some specially springy ballast between the track ballast and the masonry? Sawdust or muskeg might answer well in such a position. The T is usually more economical than the wing abutments, and a wing abutment with a heavy bank behind it is practically bound to crack at the angle between the body and the wings, and the span between parapets to be materially reduced by the tilting forward of the body; these movements, however, do not continue long, and disasters do not often result, though the writer knows one division engineer who built his abutments on the plans from the head office, and then refused to fill behind them, alleging that they would be overthrown by the bank pressure, a statement that was in a short time proved on the next division; they, however, indicate that the inability of the parts of the wing abutment to act together detracts seriously from its usefulness, and that the T, if it be proven to stand traffic when properly protected, is the preferable form, being cheaper, and not liable to change in position or cracking. The writer knows one series of structures that carry a traffic of about twenty trains a day, some of them containing seventy-five or more cars; their water-ways are arches varying from ten to forty feet span, the length of the masonry along the barrel is ten feet, the spandrel walls are six feet from out to out, and are corbelled out to eleven feet at grade; these structures have long carried their heavy traffic and the spandrel walls are in perfect repair. The life of these structures would not indicate that traffic hammer was very destructive on good masonry. Their designer appeared to have an antipathy to wings, and wherever he was forced to construct them planned them so that practically no pressure could come upon them to crack them off from the body of the structures, such a plan necessarily making them very expensive.

The writer must admit that in individual cases the majority of the questions raised by him are of minor import, yet the frequency of their recurrence makes a clear knowledge of them of great value to any engineer, and ignorance of such simple practical questions has before now at times ruined the reputation of well-trained and highly educated engineers who have neglected to protect their employers' interests by watching the small economies. This tendency is perhaps more marked among the specialist staffs of the great systems than anywhere else. The writer has not attempted to enter freely upon a discussion of any one of the questions brought forward in this paper, and has refrained in most cases from entering into any statement of his own opinions upon them, hoping rather by the presentation of such a fairly lengthy list of familiar questions to call forth a full and free discussion of them at the Society's meeting, in which he may take his part, and express such opinions as he has upon the points in question.

DISCUSSION.

Mr. Kirkpatrick

Mr. A. K. Kirkpatrick :— Transition curves are not required, in the speaker's opinion, on curves of 1637 feet radius and greater, as up to this radius, if the curves are centred to a true centre and the elevation of the outer rail is carried to the point of curve, and then depressed at the rate of 1 inch to every 90 feet along tangent, until it comes to the level of the outer rail, easy riding will result up to a speed of 50 or 55 miles per hour. One inch to the degree at this speed, in the speaker's opinion, has been found sufficient. The principal cause of rough riding on curves is that they are not centred properly, there being flat and sharp spots all through the curve, on account of the sectionmen lining the track by the eye.

The line should be thoroughly ditched during construction, both through cuttings and along embankments, and all ditches laid to a grade to carry off the water. Great care should be taken that no foreign water is brought into any water course or farmer's line ditch, as thereby endless suits for damages will be avoided. Each culvert or opening should take only what water naturally flowed in that direction before the road was built.

Where trestle bents rest on a pile foundation, the type with centre posts will, in the speaker's opinion, be easiest to keep up. In maintaining four pile bent trestles he finds it necessary to drive a fifth pile, — that is, one in the centre between the track piles. Also in renewing pile bent trestles he uses five piles to the bent, as it saves times and labour and does not delay traffic, as track piles are driven just outside of the rail and the centre one between the rails, and the track is not disturbed. With cedar piles it is almost impossible to drive a four pile bent so that it will stand without settlement under the present long and heavy trains, without brooming or splitting the piles.

The fastenings as generally used are a great source of annoyance to the bridge gangs and expense to the company in renewing trestles. The mortice and tenon on the sill at the foot of the posts rots very quickly on account of water lodging, even when a drip hole is bored through the sill from the bottom of the mortice. Flat straps $2'' \times \frac{3}{8}'' \times 2'$ spiked on with 8″ ship spikes, answer well, and are easy of removal to repair or replace the posts. The caps, in the speaker's opinion, should never be drift-bolted to the posts, nor the stringers to the caps. The top of the posts may be tenoned into mortices in the under side of the cap, but straps will serve just as well. The deck should be fastened to the posts by a strap bolt, the bolt passing through the outer guard rail, tie and jack stringer and the strap part spiked to cap and post. The nut on the top of the bolt should be secured by a pocket lock washer. The track stringers if toe-spiked to the cap at every second bent will prevent any longitudinal movement of the stringers.

The speaker differs from Mr. Kerry as to the merits of the shoulder or dap being greater than the possible damage caused by the splitting of the stick from the corner, having had occasion to remove quite a number of timbers for this cause, *i.e.*, cracking from corner of dap. If the timber in the stringer is of perfectly straight grain, there is very little liability of the stringer cracking; but if the grain is twisted, or there is a knot in the vicinity, then the stringer is almost sure, in very dry weather, to split from the corner of the dap. If the tie immediately over the cap is let down into the stringers half an inch, and with a system of inner and outer guard rails sized down one inch on the ties, and jack stringers bolted through the ties to the outer guard rail, the tendency of stringers to creeping is reduced to a minimum and the strap bolts and toe spiking prevent any movement.

The wide deck (14 ft.), the speaker believes, is an easier riding one on account of the jack stringers being farther from the centre, preventing oscillation of the train in passing over, and that they are more than an ornament is proved as the speaker has had to replace a number of cracked and broken ones. The fourteen foot deck with 10″ × 10″ outside guard rails, 5″ × 8″ inner guard rails, 8″ × 8″ ties spaced 4″ apart and 9″ × 15″ stringers at present prices for timber costs about $2.50 per lineal foot of deck for timber, $1.10 for labour of removing old deck and framing and putting on new deck, and about 25 cents per foot run for iron.

In this Northern climate, trestle bents on mud sills are sure to heave more or less during severe frosts, and throw the track out of line and surface. The re-spiking of the track into line soon cuts the ties so that their life is reduced by two or three years. If the sills are covered up sufficiently to prevent the frost heaving them, the posts are soon destroyed by rot at the ground line. The banks should be built out full, to at least half a bay beyond the end bent. If this is not done there are sure to be two or three track ties at the approach to the trestle hanging by the spikes to the rail, as this, the point between the yielding surface of the bank and the rigid trestle, is found by sectionmen to be one of the hardest points on the track to keep in surface.

The speaker thinks that 2 feet between the covers of a box culvert and the under side of the ties is sufficient to prevent covers being broken by the hammer blows from an engine. He has seen 12 in. potsdam sandstone covers on 3′ × 3′ culverts which have stood for over thirty years with from 18″ to 2′ of ballast between the ties and the covers, and are good to-day. The speaker's experience with culverts with light covering is to find a depression at these spots during severe freezing weather, necessitating placing "shims" between the ties and the rails to bring it to a surface, on account of the track on either side of the culvert being heaved by the frost.

The speaker thinks that it is advisable in this Northern climate to lay all stone box culverts in cement mortar, as the dry stone walls are liable to be crowded in by the action of frost. In dry stone box culverts that stand full of water for any length of time, and are covered by a bank of soft clay of a sandy nature, the material in the bank is liable to be washed through spaces between the stones, and cause a sag in the track.

Prof.C.B.Smith Mr. Smith:—Mr. Kerry has thrown the burden of this discussion on the railway maintenance Engineers, and there is little that one, whose experience (like Mr. Kerry's) has been chiefly on construction, can bring forward in answer to his long list of queries. There are two points, however, on which a generality may be ventured, and they are: (1) custom, (2) physical and climatic differences which form the basis of these customs.

For instance:—In classification, some localities offer no difficulties worth mentioning. Solid rock, boulders, etc., are easily definable, and a rigid statement of three or four classes can be fairly well made and adhered to. If possible, this is the best course to pursue, for Resident Engineers cannot always be found having experience, judgment, and honour sufficient to entrust in their hands a sliding scale of classification supposed to cover reasonable cost plus reasonable profit. This is the general method pursued in large sections of the United States, and the reason is not hard to find. It is the peculiar grades of material met with, which range all the way from hard pan to solid rock through a dozen indefinable stages; thus the custom has become established, even where it is not needed. Again, the practice to trestles, narrow and wide decks is largely a matter of custom. No one can demonstrate to a nicety whether a narrow-decked trestle, such as is used by most of the United States railroads, is or is not as safe as a wider one. Probably the use of flaring guard rails at the ends and a Latimer re-railing guard will make a trestle with guard rails seven feet apart as safe as a wider one with double guards. The passage of snow plows has probably forced a wide trestle on the northern part of the Continent; but in Pennsylvania or south of that, the narrow deck prevails, also in the West and Southwest.

With reference to the foundation of the end bents of trestles, there are, in the speaker's opinion, only two methods that can be at all commended,—one is to drive piles through the bank, after it has been built, into the solid ground beneath, and then erect the bents clear of the bank and free from liability to rot. The other is to build a terraced bank, and on each terrace rest a bent on a wide floor of mud-sills. The latter method gives lots of work to the bridge gangs for a few months in re-lining and re-surfacing, but utimately a firm foundation is obtained, and the trestle is, at all times, free for repair and easily surfaced. The choice between these two methods will probably depend on the soil, a bad clay or mica sand demanding piles as the only safe method.

One point which Mr. Kerry has not mentioned is whether decks separated by deck-stringers, or a continuous cap-sill method with girts or walings, is preferable. The former, although a little more expensive, has many advantages from a construction point of view. (1) In erection. The deck-stringers form a floor on top of each deck on which to erect the next one. (2) The trestle is moderately safe, during erection, from being blown down by gales. (3) It gives much greater rigidity if properly boxed both on cap and sill and on the deck stringer itself. From a maintenance point of view it would appear that repairs would be facilitated.

In fastening posts, the best combination the speaker knows of is two dowel pins in the bottom, with a flat boxing and mortise and tenon at the top, the development of rot at the bottom mortise is avoided and the post can be easily erected or replaced.

Closer spacing of ties is the decided tendency of the day; ties should not be more than 4″ clear, so that the wheels if derailed have little tendency to cause "bunching."

Elevating the road bed on curves below the ballast has many commendable features. It economises ballast, and also, what is more important, presents a proper roadbed for the track before ballasting is done. The cause of many bent rails which never can be straightened is the rapid running of ballast trains on the unballasted roadbed, where many ties are imperfectly shimmed up, on the outside of curves, with flat stones. A much more important item, however, is drainage. In the case of embankments, the tendency is for the top to become rounded ; but in cuts, the reverse is the case, and it does not matter how much or how good ballast is added. If there is a low place in the centre of the roadbed water and a bad spot in the track will result. Cuts should, therefore, in addition to thorough ditching of the road bed, be given a camber of two or three inches. The proper person to superintend berme ditching is the resident engineer ; he has time and knowledge greater than the track foreman, into whose hands such matters afterwards fall, and a thorough and continuous berme ditch along the upper side of cuts and fills, leading all the water directly to the culverts, is the only thing which should be thought of. Anything less than this means that the cuts will be washed down and the cut ditches filled up, during the first half year or so of operation in spite of the best efforts of the maintenance gangs.

Mr. D. A. Stewart.

Mr. Stewart :—This seems a large list of subjects for one paper, and the writer will not attempt to discuss them all.

Classification.—The variations in classification may be wide enough, but even greater differences in prices than those quoted by the author may occur without the classification being at fault. For instance, good slush scraper work under favourable circumstances may pay well at 10 cents, while bad clay under less favourable circumstances may be a losing job at 30 cents, yet scarcely anyone would class either but as earth. But there is no doubt that the classification of mixed material is one of the most difficult problems that meets the engineer on construction, and the writer thinks that if the line has been finally located before bids are called for, and ample time can be given to intending contractors to examine the work, a division of the grading into earth and solid rock only would work out more satisfactorily than any attempt to specify all the varieties of material that may be met with. But if the location is not final, or there is not time or opportunity for the contractor to examine the work, a threefold classification is the fairest to both sides, and as it often happens that earth changes to mixed material by insensible degrees, a percentage classification will in such cases be the only method by which the price paid can be made proportional to the actual cost of the work, which should be the aim of classification. While the resident engineers should watch such work carefully and keep accurate force returns, classification is a question to which division and chief engineers should give a great deal of their personal attention.

Overhaul.—This case is similar to the last. If the profile on which tenders are made shows the work as it is to be done, and the distribution of the material taken from the cuts, the contractor may be expected to make his bid to suit, otherwise overhaul should be allowed. But the distance at which overhaul begins should depend on the character of the work and the plant that will have to be used.

Transition Curves.—The need for transition curves increases with the degree of curvature, but not uniformly, because it also increases with the speed of train, which should be decreased on sharp curves. For instance, a 6 degree curve may require a transition curve of a certain length, but a 12 degree curve does not require one twice as long, because the speed of the train should be much less than over a 6 degree curve. The writer thinks that for speeds up to 50 miles an hour, transition curves are not needed, unless the main curves are sharper than 2 degrees. When recentering part of the line between Winnipeg and Fort William the practice was as follows: "Curves under 3 degrees "need not be flattened at the ends; but if in running them in, their ends "should come inside the tangents, they should be brought to the tan- "gent by the rule which follows. Curves of 3 degrees and sharper to "be brought inside the tangent one foot or over for a 3 degree curve to "2 feet or over for a 6 degree curve (6 degrees being the maximum "curvature), and to be brought in to the tangent by the following rule: "Call the perpendicular distance from the end of the main curve to the "tangent the main offset; the transition curve and the main offset "bisect each other. Measure back on the tangent a distance such that "one-half the main offset shall be the corresponding tangential offset "for a curve whose degree is one-third that of the main curve, the "point so found is the beginning of the transition curve. Divide the "distance so laid off along the tangent into any number of equal parts, "and at the points so found lay off on the side next the curve offsets "proportional to half the main offset as the square of their respective "distances from the beginning of the transition curve. From the end "of the main curve lay off on the curve the same distance as on the "tangent, divide it in the same way, and lay off the same offsets on the "side next the tangent; the series of points so found will form the "transition curve." The curvature of the two branches of the curve so laid down and of the main curve will be in the proportion 1 : 2 : 3, which will do well enough for most cases.

This was on a completed road where the cuts and banks were not wide enough to allow more variation from the line of the track as laid. The author's disposal of the first objection to the use of transition curves during construction is too sweeping. Their use will usually add to the labour of the resident engineer, who from the length of his section, bad roads, and the rapid progress of the work is often an overworked man. The second objection seems also to be a real one.

Long Curves and Tangents in contrast with a line of almost incessant light curvature. This question is too wide. The alignment should depend on the natural features of the country and the nature of the probable traffic. If the curves are light enough, and the tangents between them long enough for trains to straighten out in, they do not appreciably increase the resistance to moving trains, but curves are an objection in so far as they obstruct the view. The tangents should always be long enough to allow the cant to be run out at an easy grade, for this reason reversed curves and short tangents between curves in the same direction should not be used. It is unreasonable to suppose that, except in very easy country, long straight lines and uncompounded curves will make the best possible location.

Narrow Banks may be so because they were built narrow, or because they have shrunk, or partly both. Many banks that have been made over-width from the start are under-width in a few years. Sometimes the banks merely settle, and the trackmen narrow them by lifting the track out of the side; a settlement of one foot will decrease the width three feet. Sometimes they wash away, in dry and open country the wind wastes them. Some banks, as peat, widen as they settle. When banks are widened by putting on new material, it is sometimes a good plan to make steps in the old slopes.

Elevating Roadbeds on Curves.—This seems to be a very useless refinement, and is seldom practised.

Ditching.—The roadbed should always be thoroughly drained, but where and how to make the ditches is a matter of judgment in each individual case, and when to make the ditches is a question of circumstances. It would seem better as a rule to make them during construction, but too often on railroads in this country construction is too much hurried to allow all to be done that should be done, or the work may be finished in winter when the ground is frozen and ditches cannot be dug, or it may be necessary to lay tile drains, and the tiles are more cheaply brought in after the track is laid. Usually in any country difficult of access, work can be done for very much less after the track is laid than before. As a rule it would seem better to do the ditching when the other earthwork is going on; but when this is not possible the best solution would seem to be to have the construction period extend one year after the track has been laid, and to retain enough of the construction staff to look after it. Ditching can be carried on very conveniently at the same time as ballasting. This is another matter that should not be left to the resident engineer alone.

Trestles.—Two posts may be strong enough as far as they alone are concerned, but they do not distribute the pressure sufficiently for the caps and sills, which are weak pieces in most wooden trestles, and in high bents the unfixed lengths of the braces would be too great. Solid posts are to be preferred when they can be obtained.

In good timber, rot does not always come first in the heart of the timbers, but in the joints and at the points where they touch the ground. To make as stiff a structure the cluster type requires more and tighter joints than the solid type, and as to contact with the earth the one would seem to be no better than the other. The strength and stiffness of the cluster type depend largely on the tightness of the bolts and nuts, which loosen as the timber shrinks, and for this reason it requires more attention than the solid.

Mortices and tenons are not necessary, they weaken the timber, are expensive, and cause decay, and ought not to be used. If properly framed so as to have a close and uniform bearing, the feet of posts are not nearly so liable to be displaced as some engineers seem to think, and in single deck bents a drift bolt through the cap and into the post, and toe-spiking the foot of the post to the sill will hold them well enough. When there is more than one deck the horizontal braces should project not less than six inches past the bents they connect, and should be notched deeply to receive both the post and the sill; the sway braces will hold the cap and the top of posts and the sills and bottoms of posts together, and generally by using care in putting on the bracing the structure can be sufficiently tied together without any mortices.

From a maintenance point of view stringers should be of only one span length, the arguments for stringers covering two spans have not much real force. As to their carrying a train over a washed out bent, it seems a mistake to try and suit a design to a case that ought not to happen, and then one cannot tell which bent is going to wash out. Only one size and length of stringer should be used on any working division, the engineer on maintenance will appreciate this when he gets word that a stringer is cracked, and does not know which of the different sizes he may be carrying to send out to replace it. When narrow stringers are used so that there are, say, six to eight on a span, stringer or chord bolts do well because they allow fewer drift bolts to be used, and allow all the stringers composing a chord to be jacked up together when making repairs, instead of having to raise each separately.

Stringers should not be notched on to the cap, as this tends to start cracks. Resistance to an endlong movement of the bents should be provided for by horizontal struts or by walings well notched on to caps or posts, and by sloping braces from bent to bent. The latter are especially necessary on high trestles on grades. On a cheap road with light traffic and a large proportion of trestle work, it may be allowable to economise by cutting down the floors of trestles to the lowest point; but on most roads this would be very poor economy. The ties should be as long as the top width of the road-bed and strong enough and close enough to carry a derailed truck, and be securely fixed against bunching. Jack stringers should be used, or the ordinary stringers spread so wide as to support the tie out nearly to its end. Ordinary trestles in a cold climate are almost sure to be thrown out of line more or less by the frost, and it is better not to notch any of the ties on to the stringers, as in that case there is no way to line the track but to change the rail spikes, which soon spoils the ties. Lining spikes driven into each fourth tie or so will hold the ties from sliding on the stringers. The heads of these spikes should be left high enough to let a claw bar get hold of them. There should be good guard rails well notched down on the ties and bolted to them.

The best way to connect a bank with a trestle is to build the bank with flat end slopes, say one-half the natural slope, and if it has steps at horizontal distances equal to the trestle spans, so much the better, and then to leave it till it is done settling before the trestle is built; but this can seldom be done, because there is not time, and as it is obviously impossible to build a stable structure on a moving foundation, the engineer of maintenance will have to make the best of it. But to make the best of a bad job, the ends of the banks should be built at a slope considerably less that the natural slope, and stepped, and the banks left to settle as long as possible. The piles should be well driven, if they can be driven through the bank and into the undisturbed earth below; though they may move horizontally, they will not sink so much as if they are driven only into the bank. And the bank bents should not be connected to the intermediate part of the trestle, but each part built and braced so as to stand as a tower by itself; by this means the inevitable movement of the bank bents would be prevented from distorting the intermediate bents. After the banks have done settling, the bank bents may be cut off and placed on a floor of mudsills stepped into the bank, and so restored to their proper position, and will then often remain without any further appreciable movement. When the banks have settled, or when the piles can be driven through them, the ends of the trestle should be on pile bents; but when this cannot be done, perhaps mudsills are as good: either is bound to go down, and the stringers will have to be raised over and over again until the settlement has ceased. Scrimping the banks at the ends of trestles is a very bad fault, but a very common one, and is aggravated by the ends of the banks being built at a steeper slope than they will stand.

Waterways.—The best and, in many cases, the only practicable way to determine the proper sizes of waterways is to watch the streams at the highest floods. A straight and unobstructed part of the channel should of course be chosen, and as it is often necessary to straighten a part of a stream at or near the crossing, this should be done as early in the course of the work as possible, as this will give easy opportunities for observing the real quantity of water.

In settled countries all existing structures should be carefully noted. In unsettled and wooded countries the labour of fixing drainage areas would be endless, and there are too many disturbing conditions to make it a safe method, except as it may be used with others. When roadbeds are hurriedly built so that only one season, or perhaps part of a season, is available for observations, mistakes are bound to be made; this is one of the penalties to be paid for haste, it is also an argument for temporary structures. The pity of it is that after the difficulty has been got over for a time by building temporary structures, the maintenance staff may be so neglectful as to be found, after ten or more years, with less knowledge of these matters than the construction engineers learned in their one year. Small hollows should, however, always be approximately measured and their flow estimated; sometimes unnecessary expense is incurred by making culverts for such places so large that no possible rainfall could fill them, and sometimes no outlet at all is provided, and banks are washed out in consequence. Gross errors are more frequently made in these small cases than in large ones. In building embankments across flat ground, it should be borne in mind that water that may pass off insensibly as a thin sheet, and without any defined channel before the bank is built, may, when obstructed by the bank, require openings of considerable size to carry it off.

In deciding all such questions as those raised in this paper and many others, the engineer on location or construction is at a great disadvantage unless he has had some experience on maintenance.

Mr. H. Irwin. Mr. Irwin :—The question of *classification* has been thoroughly threshed out in *Engineering News* quite recently in a series of articles relating to the Chicago drainage canal, so that there is very little more to be said on the subject. All material to be excavated should be classified as earth, loose rock and solid rock ; all boulders of over half a yard to be counted as solid rock, and all under half a yard but over—say two cubic feet—to be classed as loose rock. This classification, of course, renders it necessary for an intending contractor to examine the ground carefully, and implies that proper test pits have been sunk and borings made, which seems to be the proper course to be adopted in all cases, as contractors can then bid intelligently. No engineer should be allowed to make a varying price on different parts of the work, as this price might vary according to the capacity of the engineer to receive and of the contractor to give "boodle." The specification should be fairly and clearly drawn up and its terms strictly enforced.

As to *overhaul*—that might be discarded altogether, if contractors were obliged to examine the ground before tendering, except in the case of borrowing ordered after the contract has been signed. As to springs and slips, contractors should make such allowance for them, before tendering, as they think proper.

Transition curves of some sort seem very necessary in the case of all curves sharper than 2°, and they can be laid out at first quite readily by Professor Crandall's method, which is the same as that advocated by Messrs. Wicksteed and Lordly ; though to Professor Crandall is due the credit of having published a set of tables to aid in field work.

The proper *drainage* of the roadbed and right of way is often badly neglected. In unsettled country the matter is generally one of simple engineering, only calling for careful examination of the adjoining country. In passing through farm lands, however, the legal aspect of the case has also to be attended to, and care should be taken to make all ditches and culverts low enough to provide for possible future deepening of farm ditches, and to see that a large volume of water is not discharged into some farm ditch which cannot quickly carry it off. As a rule, the drainage system of any farming locality should not be interfered with, except under sanction of the local ditch inspector, or after proper written agreements have been signed by all interested parties. Neglect of these precautions in the past has led to numerous law suits, entailing vast expense, annoyance, loss of time to railway officials, and general bad feeling on the part of farmers towards railway companies.

As to *trestle stringers*—it would be interesting to know exactly the number and dimensions as well as species of timber of the stringers which carried a train across a double span where a bent had been washed out, and the span of the double bent with wheel diagram of the locomotive.

With regard to *dry stone box culverts*, they allow water to soften the bank, and if blocked would be much more likely to cause a washout than culverts built in cement with proper wings and vertical sheeting below the floor at the ends.

CORRESPONDENCE.

Mr. D. MacPherson.

Mr. MacPherson :—The author has certainly raised a number of questions, which, if definite and conclusive answers could be given to them, would be of assistance to many engineers.

With regard to *classification* of material during construction, it appears self-evident that it is absolutely impossible to attempt a classification which will literally and exactly cover all the points which may arise in even a small contract. The best practicable plan for all ordinary cases is to make the simple classifications of solid rock, loose rock, hardpan and earth excavation, defining limits for each, and then make the prices in accordance with the local value of wages and the known qualities of the different materials. The contractors have presumably made some study of the different classes of material before tendering. The cause of narrow banks would appear to be either that they were originally built so, or are composed of material that will not stand at the slope given it, and have in consequence gradually slidden down to a flatter slope. Banks should certainly be built wide enough to be of standard width after settlement of slopes.

Ditching is a subject to which ordinarily too little attention is paid, but the only general rules would appear to be to make the ditches large enough to carry the water freely, and to keep the water line at least below the level of the bottom of the ballast. In locations where, from the nature of the material in the bank, the track is liable to heave, the water line should if possible be kept as far below rail level as the extreme penetration of frost. The average roadmaster whose track heaves badly will at once call for more ballast, when, by a judicious cleaning or deepening of ditches, a cheaper and more effective remedy is in his own hands.

The solid four post *trestle* for ordinary heights has been used exclusively by the writer, for the reason that large timber is plentiful on his division.

Drift bolts make the most satisfactory all round fastening, because they are cheap in first cost, and do their work well; though they are undoubtedly difficult to remove for minor repairs.

The "stirrup" fastening mentioned by the author would appear to be of merit for fastening caps to posts. The standard main line trestle used by the C. P. R. has bents 13′ centres with two 9″ × 15″ × 14′ stringers under each rail in one bay, overlapping three 6″ × 15″ × 14′ stringers in the adjoining bay. These overlapping stringers are bolted together and drift bolted to the caps. In addition to this there are 9″ × 15″ × 14′ "Jack" stringers on each bay. Ties are 8″ × 8″ × 14′ spaced 12″ centres, and inner guards 5″ × 8″, outer guards 8″ × 10″.

These closely spaced long ties, supported almost to their ends, have in several instances safely carried derailed trains, and in no case has a wreck occurred through bunching of ties on a standard trestle. It appears essential to both fasten the stringers to the caps, and some of the ties to the stringers; otherwise a derailed truck slewed sideways will either bunch the ties or pull the stringer off the caps. Only very cheap trestles and bridge floors would be necessary if one could be sure of trains never leaving the rails. It takes a good floor to carry safely a derailed train. Dapping stringers, though theoretically all right, does undoubtedly in practice cause splitting of the stringer, beginning at the dapped angle. To connect a bank with a trestle properly, either piles must be used in the approaches, or the bents must be carried down to good foundations, and in all cases the filling must come some distance in from the end of structure, and be left at its natural slope after settlement, which will avoid all tendency to shoving in of bents.

As to the size of permanent *waterways*, the writer's practice has been to have all structures numbered, and definite and distinctive high water marks accurately marked on them each year, with a record in the office of these marks together with rates of current at high water. When it is desired to replace a temporary structure with a permanent one, there is thus a more or less extended record of the water flow.

Accurate sections are made of the maximum areas, and with this and a knowledge of the general topography of locality it is not difficult to decide upon a safe and economical size for the permanent waterway.

Mr. MacKenzie.—*Classification.*—The following classification of materials was prepared by the writer for the construction of the Dartmouth branch of the Intercolonial Railway, under the general direction of P. S. Archibald, the Chief Engineer. The work of excavation has been completed, and the engineers and inspectors have had little trouble in deciding as to the proper classification of the different materials Mr. W. B. MacKenzie met with, to the satisfaction of both engineer and contractor:—1st. Solid rock excavation. 2nd. Loose rock excavation. 3rd. Hardpan excavation. 4th. Borrow-pit excavation. 5th. Foundation excavation. 6th. Earth excavation.

Transition Curves.—On the Dartmouth branch, offsets for transition curves were made at the time of location, and the circular curves run in from the offset B. C's. and E. C's. It is intended that the easements shall be put in when the ballasting is being completed. The offsets were:—0.654 ft. for 3 degree curves; 0.763 ft. for 3° 30′ curves; 0.872 ft. for 4° curves; 0.981 ft. for 4° 30′ curves; 1.090 ft. for 5° curves; 1.200 ft. for 5½° curves; 1.310 ft. for 6° curves; 1.415 ft. for 6° 30′ curves; 1.52 ft. for 7° curves; 1.63 ft. for 7° 30′ curves; 1.74 ft. for 8° curves; 1.85 ft. for 8° 30′ curves; 1.90 ft. for 8° 45′ curves, the latter being the highest degree of curvature on the line. Of course the center line on every curve is out of its true position, as shown on the right-of-way plans, to the extent of the offset, which may by some be considered objectionable, where land is valuable; but if the offset for each curve is marked on the right of-way plan, and allowed for in future measurements for fences, etc., there can be no objection to the method.

Long Curves and Long Tangents.—If grades are properly equated for curvature, and sufficient tangent lengths allowed, there does not seem to be any reason why curves should not be introduced to any extent, none of course exceeding the limit of resistance fixed for that particular operating division. The writer can point to an important line originally located around a summit, almost on a level, but afterwards changed by a "straight-line" fanatic, for the purpose of making the line a straight one and saving two miles of distance; and, to-day, it is necessary, on account of the heavy grade, to lay off seven loaded cars from every fully-loaded train which can be hauled over the remainder of that operating division.

Narrow Banks on Old Railways.—Heat and cold, rain and snow will, in time, bring down slopes of both banks and cuts to 1¾ to 1, or 2 to 1, no matter how the banks are first formed. As a rule, it would be absurd to compel contractors to make up the banks full width from the start, although a clause of that kind in the specification might be a good thing for occasional use, under a common-sense engineer.

Trestles.—The four-post trestle of single timbers seems to be good enough for ordinary use up to 30 ft. in height, and the built up posts for greater heights. Mortise and tenon should not be used; dowels and drift-bolts are better and cheaper. There is not much difference in the life of different parts of a trestle, and re-inforcing pieces spiked on are generally sufficient to carry over weak and decayed members until the whole is ready for renewal. On a number of four-post spruce and hemlock trestles which had stood seven years, many timbers were bored, and it was found that the different members required strengthening as follows:—

46 % of the stringers	were partly	decayed and	required	strengthening.
62 % " caps	"	"	"	"
21 % " posts	"	"	"	"

The writer prefers corbels and double stringers, placed side by side, breaking joint and extending over two bents, the stringers being bolted to each corbel with four screw bolts and the corbel itself drift-bolted to the cap.*

The stringers should not be dapped on the cap. The writer has examined hundreds of stringers under traffic, and has seldom failed to find that a crack had started, sloping upwards from the edge of the notch, where the stringer would eventually break, if allowed. He maintains that the depth of a stringer cannot "be sacrificed at the ends without detriment to its utility." Ties should not be less than 12 ft. long. Dapped guard-timbers are of little use, as the parts between the notches split and fall off in time, and allow the ties to bunch up under derailed wheels. Spacing-blocks is the only proper method.

Connecting Trestles with Banks.—No driving of piles or placing of trestle bents before or after the bank is made up will prove satisfactory. Piles in a made bank rot very quickly, and if on sloping ground they will be carried out of line by the settlement of the bank.

After the bank is completed, carry the stringers well back over the end, and support them on a small log crib filled with stones resting on a ballast-floor. This will require some slight blocking up for a year or so, as the bank settles.

After the road has been in operation for some years, and the crib has partly decayed, the bank will be solid enough for a small stone masonry abutment, resting on a concrete foundation. First, six inches of sand should be hammered in over the bottom of the excavation, then 18 inches of concrete, having a few pieces of old rails imbedded lengthwise in it, then the masonry. The foundation pit should slope from the ends

* The writer knows of a case of a four-pile bent being carried away by floating ice, and before the accident was discovered, the continuous stringers and corbels had carried two trains safely over a gap of 30 feet.

toward the center and a stone drain lead out from there to the slope. The crib will supply stones enough for the drain, the concrete, and the filling at the back of the wall.

The Best Method of Determining the Size of Small Structures.— The minimum size of culverts should be large enough for a man to creep through, and the maximum must be decided from the amount of rainfall, the drainage area, porosity and slope of the ground. A personal examination of the ground should be made; noting of freshet-marks; height to which the water may be held back without damage to property above. Existing openings on the same stream should also be observed, and a cross section of the stream at a narrow place at high-water obtained.

Masonry for Culverts.—In the writer's opinion, dry masonry is a delusion, excepting for special cases, and, as a rule, should never be built where it is possible to use cement masonry at reasonable cost. To make good dry work, large, well-shaped stones are required, and they must have considerable work expended upon them by hammer and steel point before they can be laid to reasonably close joints; so that in many cases the cost would be, or could easily be made to be, greater than masonry laid in cement mortar.

Rough rubble stones of hard quality from cuttings, or boulders from the fields, make excellent box culverts, when roughly shaped with the hammer and laid in cement mortar; and even arch culverts can be and are so made. Cover stones for boxes may have to be specially quarried, but many of the flat stones picked up along the line can be used for this purpose. When so built, these cement masonry culverts will vent water under a head and stand up to their work, even should a cloud-burst cause the banks to be washed away.

As the duty of a cement is to convert the separate parts into one monolithic structure as well as to resist atmospheric forces, it is plain that the best Portland cement is the cheapest in the end, when labour and freight charges are considered.

Covering of Box Culverts.—A foot of gravel or sand between the covers of box culverts and the underside of the tie is the minimum thickness which should be recommended.

Tee Abutments.—Tee abutments, with twelve inches or more of ballast between the masonry and the ties, remain undamaged by traffic, but they are objectionable because of the narrowness of the roadway along the stem of the tee. The writer has seen bridge-approaches thus made so much narrower than the bridge, or roadway, that should a derailed car be dragged to this spot, certain disaster would result.

Wing Abutment.—If wing abutments have proper tie-walls between properly designed wings, there will be no cracking between the wings and the body. Splayed and stepped wings are always preferable to right-angled wings. The wing or any other abutment which cannot retain its bank, however deposited, without tilting forward or cracking, is not properly designed, and has no right of existence as an abutment.

Mr. Kerry:—The writer is much pleased that his appeal for information on those points, concerning which there is too often an enforced ignorance on the part of construction engineers, has met with such an able response. As the paper was rather interrogative than positive, the discussion has taken the turn of elaborating the facts concerning the various questions taken up in the paper, and, consequently, calls for but little rebuttal. The writer is not inclined to believe that Mr. MacPherson's fourfold classification is generally applicable; there are doubtless wide areas in which it would prove perfect in operation, but there are also other wide areas in which earth, hardpan, and loose rock are found imperceptibly grading into one another, and the most capable and careful contractor, after exhaustive examination, can only tender on the supposition that the coming engineer will classify the material in exactly the same grades as the past one has done. The ordinary question put by the contractor in such districts is whether the classification (not the specification) will be the same as it was on some branch or road on which he has previously been working. Personally the writer prefers a classification in two sections only—solid rock and everything else. Such a classification was approached in the paper, and was brought up and generally commended in the informal discussion at the ordinary meetings; however, in many instances, a more intricate classification will be found perfectly suitable, and the writer would only insist that in every case the engineers should be certain, before issuing their specifications, that every grade of material that will be met with is so clearly and definitely specified that no question as to its class can arise, and that where there are no decidedly recognizable distinctions between the various classes of material, no attempt should be made to distinguish between them in the specifications. This course will at times reduce the excavation specification to a single clause, and will throw the onus of determining the proper price for moving the material upon the contractor, to whom it properly belongs. It too frequently happens that there is no one upon the work capable of classifying material in percentages of nominal grades, the chief engineer lacking the opportunity for personal observation and the resident

Mr. J. G. Kerry

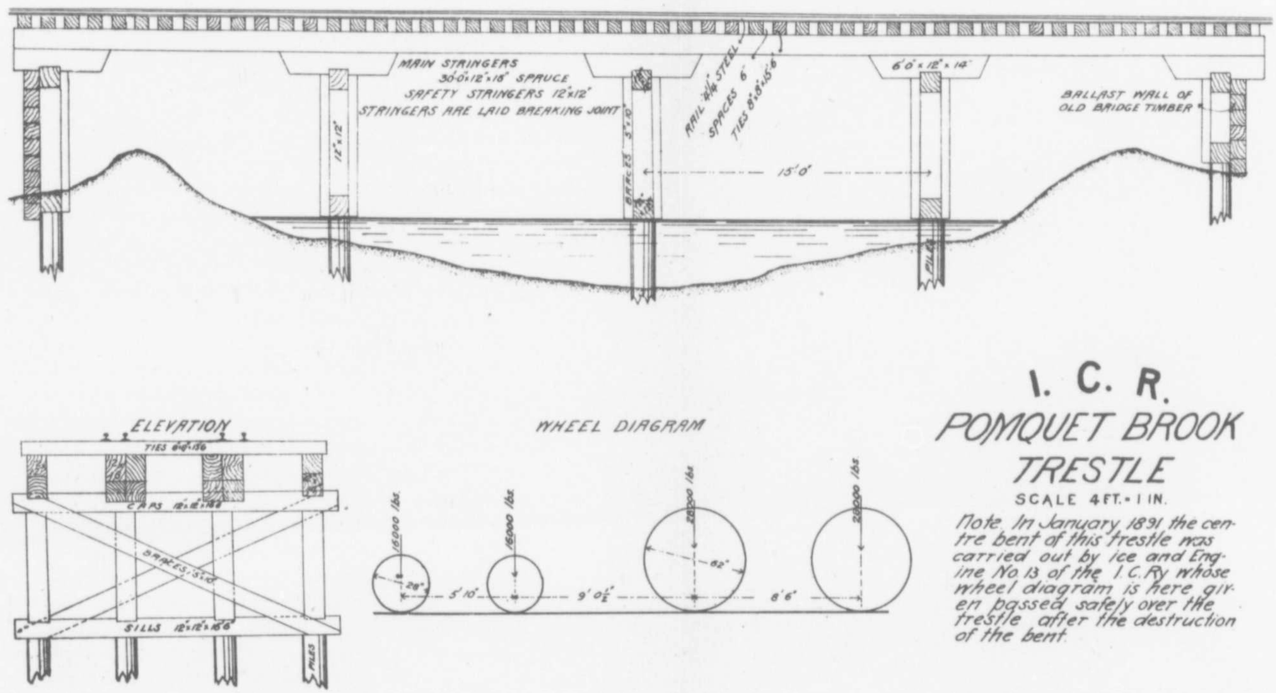
engineers lacking experience, and a definite classification resting on broad distinctions removes all the possibilities of dispute that are present in such a case. It would appear to the writer that the arrangement of stringers described by Mr. MacPherson, two 9″ pieces alternating with three 6″ pieces, would prove practically unsatisfactory owing to the fact that the 6″ pieces are very narrow in comparison to their length, and that the arrangement requires that the stock on hand for repairs and accidents shall be in two sizes. Mr. MacPherson's method of observing and recording the discharges of the streams crossing the line is excellent, and will furnish as complete information as will ever be required in designing suitable waterways for the structures. Mr. Stewart's remark, that it is unreasonable to suppose that long uncompounded curves and long tangents can generally make the best line, is very true; but even this truth does not prevent a great number of lines being so located and built. This may be due to the fact that any one can look at a plan and profile with simple alignment and be much pleased with the location and with the man who made it, but no one except an experienced engineer can observe how much needless expense the company incurs for the luxury. There is no doubt, as Mr. Stewart says, that cluster trestles require far closer watching while building, and more careful attention afterwards, than solid trestles do; the rapidity with which the nuts work loose on the bolts, especially on new trestles, is very strange and somewhat difficult to explain, and constitutes a strong objection to any structures that require many bolt fastenings. Mr. Stewart's suggestions about the connections between trestles and banks point out what is undoubtedly the main cause of trouble at such points, namely, that the ends of the banks have been insufficiently sloped. Railroad engineers set out side slope stakes, and in most instances should insist upon the banks being built out to them; but it is very rarely that end slope stakes are set, and no one ever insists upon the bank being built to them, and yet this end slope is a most important feature of good construction. The writer's suggestion regarding the determination of waterway for small structures was intended only for those watersheds whose discharges can be passed by culverts, and he recognizes that the determination of the watershed of a large stream is not of the same relative importance as in the case of a small one, because sooner or later on some occasion every part of the drainage area of the latter will send down at the same time a maximum discharge to the culvert; but owing to the limited areas of excessive rainfalls, varying conditions of improvement and the different times required for the discharges from distant points to reach the culvert, large areas cannot do so. In general, however, unless the slopes of a country be very gradual, it is not difficult to ascertain the areas of watersheds to an approximation sufficient to guard against enormous discharges out of hollows that are apparently insignificant where the road crosses them.

Mr. Irwin's discussion contains some very valuable remarks on specifications, but it reminds the writer of the old saying that adjectives have no place in engineering papers, and the main point emphasized by the variety of opinions on this head has been the difficulty of preparing specifications "fairly and clearly drawn." The general procedure in the matter of letting contracts recommended by Mr. Irwin agrees with that advised by the writer, in that they both necessitate able contractors of considerable financial strength and would disqualify that great body of railroad builders who contract with monthly estimates as their capital, and who are always ready to bid the lowest prices and prove the most useful tools to a close manager of construction in his efforts to keep down costs, for being too weak to fight against the manager's rulings, they accept them, and thus establish precedents. To the railroad companies such a disqualification would be a misfortune, but on broader and humanitarian grounds it might be right. Mr. Irwin's discussion of ditching brings up a matter that is rarely ever thought of during construction, and never appears in current technical literature. The usual procedure is to consider ditching from the direct standpoint of its effects on the cuts and fill, if it is considered at all, while, as Mr. Irwin points out in settled districts at least, the effect of the construction of the line upon the natural or artificial drainage of the surrounding land is really the most important side of the matter, and the neglect of it can only be explained on the ground that the construction department has ceased to exist before the damaging effects of the error are felt and the law-suits commence. Through the courtesy of Mr. W. B. MacKenzie, of the Intercolonial Ry., the writer is able to furnish Mr. Irwin with full details of a case where double length stringers carried a train over a washed out bent. These details will be found on the accompanying plate.

Mr. Kirkpatrick's experience with the rotting of piles and trestle posts at the ground line would indicate that, except under the conditions mentioned by Mr. Smith, bank bents built on mudsills laid on properly graded benches will prove the most satisfactory connection between trestles and banks; but the two precautions mentioned by Mr. Stewart must not be neglected if a satisfactory result is expected, and those are that the benching must be made with a flat end slope and the bank bents disconnected from the rest of the trestle. Until reading over the discussion on this paper the writer was of the opinion that the often repeated objection to shouldered stringers was more theoretical than practical, but the unanimity of the condemnation of the practice in the discussion makes it evident that it is a very practical fault. The system of fastenings recommended by Mr. Kirkpatrick is much more easily worked with than are those in general use, and its efficiency is proved by his experience. The effect of heavy frosts upon the roadbed over culverts, as described by Mr. Kirkpatrick, is precisely opposite to that reported on lines in the Middle States during the cold snaps there; but the frost there has not time to heave the whole roadbed as it does in Canada, and only freezes through the thinnest parts, *i.e.*, over the culverts. The writer has had no experience with dry stone culverts in Canada, but knows that farther south, in districts exposed to heavy frosts and frequent thaws, these culverts, when well built, have given complete satisfaction, and the banks do not become seriously water-soaked from them; the writer would not, however, recommend dry masonry for partially submerged culverts, or for any that are forced at times to discharge under a head; the greater proportion of culverts are, however, dry throughout more than half the year, and for these, dry masonry is very satisfactory.

The writer is not inclined to admit the general application of Mr. Smith's reasoning in defence of roadbed dressing; if the track could be laid immediately after the completion of the roadbed it would be useful, but settlement, shrinkage, haulage and storm soon destroy it, and it is of permanent value only when in cuts or on roads of very high curvature. The roadbed should always, however, be carefully dressed in so far as is required to protect against the formation of hollows and pools under the ballast. The deck stringer system for double deck trestles, which is recommended by Mr. Smith, is objectionable practically, on account of the great number of surfaces of crossing pieces it brings in contact, and which surfaces serve as a ready starting point for decay.

The writer does not agree with Mr. MacKenzie's statement, that it would be absurd to compel contractors to make their banks full width from the start. Under certain conditions, such as train filling, this may be true; but in ordinary grading by cart or dump car, it can be done with ease, and the writer has seen so many instances of damage caused by not observing this rule that he decidedly recommends it. On a large



I. C. R.

POMQUET BROOK TRESTLE

SCALE 4 FT. = 1 IN.

Note. In January 1891 the centre bent of this trestle was carried out by ice and Engine No. 13 of the I. C. Ry whose wheel diagram is here given passed safely over the trestle after the destruction of the bent.

road where a permanent carpenter's gang is maintained, a system of repairing by putting in new pieces in place of decayed ones would seem to the writer to be preferable to the system of reinforcing pieces recommended by Mr. MacKenzie; of course, the trestle fastenings should be designed so as to admit of a ready removal of any piece without interference with the traffic; the repair timbers will last longer, as they are not placed in contact with timbers already decayed, and there is not the same tendency to allow a trestle to outlive its usefulness as there is when a whole trestle or none must be replaced. The stone-crib base for connecting trestles to banks has never been seen in service by the writer, but is of excellent promise, and could be used with proper benching for all bank bents, and would not decay as rapidly as mudsills, but will probably cost more in the first place. Mr. MacKenzie's argument against dry culverts holds good in districts where quarrystone is scarce and means of transportation good; but culverts built as he describes are inferior as masonry to dry masonry, because such a mass of unshaped stone has no bond, and would not stand at all without mortar. In the extreme case for comparison, a well built concrete culvert is altogether superior to dry masonry, but where transportation is bad and quarry stone abundant and easily worked, a dry masonry builder will put up a well built, well bonded culvert out of large and well shaped stone for considerably less cost than any cemented structure can be built for, and it will prove perfectly satisfactory under the ordinary conditions of culvert service. The writer has seen this class of masonry built for $3.50 per cub. yd., where concrete and cemented masonry cost from $5.00 to $5.50. Mr. MacKenzie's decision on the cement question is too sweeping. As remarked in the paper, the writer is perfectly willing to admit the intrinsic superiority of the Portland cements; he is also perfectly willing to admit the superiority of granite masonry to limestone masonry, but would not, on that account, reject limestone masonry when its price is lower, and, with the same reasoning, does not think it wise to decide what variety of cement to use on considerations of unit strength alone The materials in every part of engineering structures can be replaced by stronger and more enduring materials, but it is universally recognized that it would not in general be wise management to so replace them, and were the writer in a position where the cost of a good brand of natural cement was markedly less than that of Portland, he would not feel justified in draining the company's resources for the latter for ordinary masonry; but, for special structures, and under water work, it would often be necessary to employ the finest cement obtainable. That there are brands of natural cement, cheap, well made, and perfectly reliable, is well known to anyone who has seen much practice in the United States, where their value has been abundantly proved by long experience. It does not appear to the writer that T abutments must necessarily be built narrower in the tails than the width of the bridges that rest on their heads, and, therefore, the type should not be condemned for the fault of some poorly designed examples. Mr. MacKenzie's condemnation of the greater number of existing wing abutments does not seem to the writer to be justifiable. It is well known that in most cases a settled bank presses but little on the abutment in front of it, and if, with a little care, the bank, as it rises, can be kept in the condition of settled earth, and so bear but little on the masonry, it seems to the writer to be but waste of money to design the masonry to bear more than that little with a fair factor of safety. Prevention is always preferable to cure. An abutment cannot fully settle upon its foundation until its load has come upon it, and, in so settling, it may show small cracks and movements; but these movements are entirely another thing to being dangerously weak, and the writer has frequently seen abutments that have moved give perfectly satisfactory service after the settlement stage has passed, and considers that those abutments were well designed, and cracked owing to some yielding of the foundation, and were he to build others in their places he would only watch to see that the filling was more carefully done, and would make no alteration in the plans. It is questionable if any wing abutment (not including U's) was ever built which did not yield to some extent under its load.

In conclusion, the writer desires to thank those who have contributed to the discussion, and trusts that the information therein embodied will prove of much value to those members of the Society who may be connected with this branch of engineering.

Thursday, 24th October.

THOMAS MONRO, President, in the Chair.

The discussion on Mr. Kerry's paper on "Some Open Question on the Minor Problems of Railroad Building" occupied the evening.

Thursday, 7th November.

THOMAS MONRO, President, in the Chair.

The discussion on Mr. Kerry's paper on "Some Open Questions on the Minor Problems of Railroad Building," and the transaction of other business, occupied the evening.

Thursday, 5th December.

THOMAS MONRO, President, in the Chair.

Paper No. 108.

A NEW METHOD FOR THE DESIGN OF RETAINING WALLS.

By W. BELL DAWSON, MA.E., ASSOC. M. INST. C.E., M. CAN. SOC. C. E.

The method proposed is to take as a basis a wall which supports an unlimited slope of loose material at its own angle of rest, and to bring all other cases into relation to this. The advantage of approaching the problem from this direction is, that the formulæ become so simple that the comparison between different forms of walls can be made with facility; and from these a wall of any required stability to meet existing conditions can be conveniently and correctly deduced.

To show at the outset that the basis proposed is not so far from natural conditions as is usually supposed, the circumstances which first directed the writer's attention to this method may be briefly mentioned. Along the Fraser and Thompson Rivers in British Columbia, on the line of the Canadian Pacific Railway, there are stretches of some miles at different places where very long slopes of sand and gravel occur. These slopes extend from near the edge of the river to a height of from 300 to 500 feet or more, where they reach the top of a terrace. They are usually more or less covered with grass and sage-brush, although some broken gaps also occur which are termed gravel-slides. The general location of the railway along the face of these slopes is at about half the height between the river and the top of the terrace above. Although this class of country has in general an irregular appearance, yet a careful examination of these long slopes shows that they are exceedingly uniform in their inclination where the material is sand and gravel; and this inclination does not vary appreciably from 33° 41′, or $1\frac{1}{2}$ to 1, the well-known angle of repose for material of this kind. There also occur long natural slopes of rock debris in the more rocky parts, which show with great uniformity an inclination of 37° to 40°, or $1\frac{1}{4}$

to 1. These are thus examples of natural "surfaces of equilibrium" such as are seldom met with on so large a scale.

These slopes, while maintaining their general inclination as the side of the main valley, are more or less fluted or corrugated on an immense scale by the valleys of the side streams. In these circumstances the railway cannot follow a natural contour without bringing in an excessive amount of curvature; but in making a series of cuttings and embankments on this side-hill ground, a practical difficulty at once arises, as the slope of both cutting and embankment would be parallel to the natural surface for a vertical height of 200 or 300 feet both above and below.

As the writer had occasion a few years ago to make estimates for the completion on a permanent basis of the railway works on 125 miles along these rivers, it was at once evident that the construction of retaining walls to support the slopes of both cuttings and embankments would be the most economical method to pursue. In designing such walls for stability, the pressure which they have to withstand is practically the same as for an unlimited slope at the angle of repose, which shows that this condition is no mere theoretical consideration.

The complexity that attaches to the problem of the stability of retaining walls arises from the fact that authors have usually considered the case of some arbitrary limiting surface for the earth to be supported, instead of the natural surface of equilibrium. The simplification of the formulæ which results in this case is sometimes touched upon as a theoretical corollary of no practical value. But the consideration of the surface of equilibrium, inclined at its own angle of friction to the horizon, brings the question into relation to the hydrostatic problem of walls for the support of water. The simplicity in their case is due to the fact that for them a surface of equilibrium is always considered.

The use of walls to support a horizontal surface of earth on a level with the crest of the wall itself (known on railways as grade walls) is so frequent in practice as to require special consideration, and to deal with this case satisfactorily certain arbitrary simplifications have to be introduced. But for walls with any considerable surcharge, which come nearer to the natural conditions as already explained, the best method will be to design first a wall of equilibrium which is at the limit of stability when supporting an unlimited surface at the angle of repose; then to allow the difference between this and the actual surcharge as a margin of safety; and if necessary in addition, to increase the wall itself for greater stability. The wall of equilibrium can be calculated without any assumption or simplification of the actual conditions, and the margin or increase for stability which there is to count upon, becomes quite distinct. It is possible also, in dealing with a surface of equilibrium, to calculate the stability of dry masonry walls, or even a heavy layer of rip-rap for the support of earthwork.

The term "loose material" is preferable to "earth," as it avoids conveying the impression that there may be cohesion in the material; as any cohesion at once vitiates the theory. Cohesion, too, may not always be on the side of safety, especially in the case of water-soaked material, which by itself may possess a certain amount of cohesion, and may yet exert an increased pressure on the wall. The term "loose material" is also the most general, and may be taken to include sand and gravel, loose rock, or even corn stored in an elevator.

It is also essential to distinguish clearly between the modes in which a retaining wall may fail, in order to know definitely the conditions under which the various formulæ are applicable. These are :—

(A) Failure by overturn around the outer edge of the base. This is the mode of failure which is generally taken for granted; but authors seldom point out what is really implied by this condition. It means physically that the surface on which the wall stands is incompressible, inelastic, and in fact absolutely resisting. This may be closely true for a foundation on rock or other very hard material. But if the surface has any elasticity, it is easy to show from the theorem of the "central third," that the resultant of the reaction against the base will have its point of application at two-thirds of the width from the back of the wall, instead of at the outer edge. With ordinary forms of walls, this will decrease the stability to about one-half; and it is a question how far this should be taken into account for walls standing on pile foundations or on timber platforms, where the elasticity may be quite appreciable.

(B) Failure by sliding on the base, or at the level of any course in the masonry. This is often referred to; but is of less practical importtance, because the weight of ordinary walls gives them a greater resistance by friction than by overturn; and if the friction is found deficient, it is more economical to adopt some expedient for increasing it, such as inclining the foundation courses against the direction of the pressure, rather than to increase the dimensions of the wall itself.

(C) Failure by sliding at an angle through the mass. This can only occur when the wall itself is of loose material; as, for example, loose rock supporting earthwork by its superior weight and greater angle of friction ; a case which we will consider further on.

In this paper, failure by overturn in the ordinary way will be considered, unless otherwise stated.

Wall supporting an unlimited extent of loose material at the angle of repose.

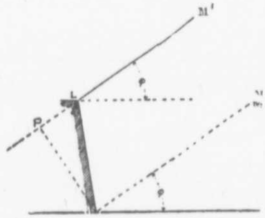


Fig. 1

Let A L represent the back of the wall, and L M' the surface of the loose material extending to an unlimited distance at its own angle of repose ϕ. It is evident that the pressure against the wall is due entirely to the layer between the lines LM' and AM, as the remaining material below the line AM is already in equilibrium without the aid of the wall.

In finding a formula for the pressure against the wall, we obtain the simplest expression by taking the value of the horizontal component of the total pressure, usually called the horizontal thrust; and this thrust is also the most convenient in finding the moment of overturn. In order to avoid mathematical detail, the proof of the formula is given in the Appendix. A graphical method of general application is there given, from which the proof is deduced ; and the simplifications which obtain in the case considered are thus easily traced. The result is also shown to correspond with formulæ which Rankine gives for special cases.

The resulting formula is as follows :—

Let $T =$ the horizontal thrust.

$w =$ the weight of the loose material supported, per unit of volume.

$p =$ the perpendicular distance A P from the foot of the wall to the surface of the loose material ; *i.e.*, the thickness of the layer supported.

Then $T = \frac{1}{2} w p^2$

It will be noted in the proof that the elimination of other variables enables the following statement to be made :—

When a wall supports an unlimited surface of loose material at an angle of repose, the horizontal component of the pressure on the wall is independent of the existence or non-existence of friction between the loose material and the wall. Its amount depends only upon the weight of the loose material per unit of volume, and upon the perpendicular thickness of the layer supported. It is also independent of the inclination of the wall, provided that the thickness of the layer supported is unchanged. Its point of application is at one-third of the vertical height of the wall.

Thus in the following figures, in which the thickness p of the layer supported is the same, the absolute value of the thrust T will be the same for the various inclinations of the back of the wall ; but the point of application will be at a variable height above the *horizontal* plane through A, and the moment of overturn will increase accordingly.

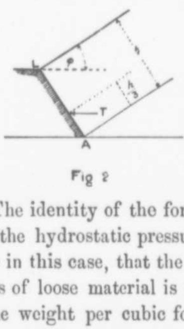


Fig 2

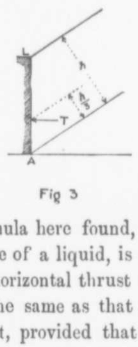


Fig 3

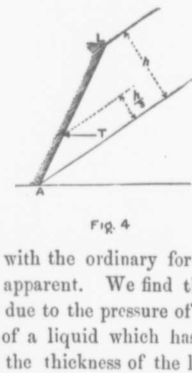


Fig. 4

The identity of the formula here found, with the ordinary formula for the hydrostatic pressure of a liquid, is apparent. We find therefore in this case, that the horizontal thrust due to the pressure of any class of loose material is the same as that of a liquid which has the same weight per cubic foot, provided that the thickness of the layer supported is the same as the vertical depth of the liquid. The only physical difference is that the loose material must have a surface of unlimited extent, while the pressure of the liquid is the same with a limited surface area.

Walls of Equilibrium.

We may now proceed to investigate the most economical forms of " walls of equilibrium," or walls which are on the point of overturning when supporting loose material of unlimited extent at its angle of repose. It is evident that a triangular form is the most economical. (As ALC in Figs. 21 and 22.) In the case of a rectangular wall for an infinite surcharge, in the series recommended by Poncelet, the base is 0.934 of the height, while the stability is only 1.86, which shows at once how far such a section is from being economical. If in Figs. 21 and 22 we adopt any given face batter (angle δ), and proceed to calculate the required back batter (angle θ), the general formula is a quadratic in terms of *tan* θ, which we give with proof in the Appendix (IV). The reason of the degree of complexity which this formula presents is that the thickness p of the layer supported by the wall is itself a variable as θ varies.

The forms in Figs. 5 and 6 are calculated by this general formula, and they represent two typical cases of sand and gravel supported by masonry walls :—

Firstly, an unlimited slope EF at $1\frac{1}{2}$ to 1 at a vertical height AB_1 above the point A. This is a very usual case in practice, as the point A may be the outer edge of foundation on side-hill ground, or the inner edge of a railway formation or road-bed, and the slope EF has to be supported at some point between B_1 and B_5. For comparison the height AB_1 is taken as 10 units, and the areas of the cross-sections are then as follows :—

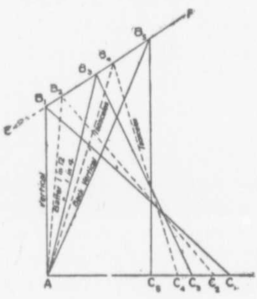


Fig. 5

Height $AB_1 = 10.00$

Area	$AB_1C_1 = 56.50$
"	$AB_2C_2 = 55.38$
"	$AB_3C_3 = 53.10$
"	$AB_4C_4 = 51.64$
"	$AB_5C_5 = 47.02$

Secondly, the wall to be built on a natural surface GH at $1\frac{1}{2}$ to 1, and to support an unlimited slope EF at a height AB above A. Here EF may be taken to represent the slope of an embankment on side-hill ground.

In both these cases the wall with a vertical back is the most economical, and it also requires less excavation for its foundation.

The following table of triangular walls of equilibrium shows the values required for the face and back batters, in some of the more

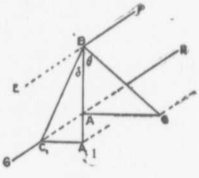


Fig. 6

Height AB = 10.00

Area ABC = 56.50

" A^1BC_1 = 47.02

usual cases. The rip-rap is considered to be in equilibrium around the outer edge of the base as in the case of masonry, and the negative sign indicates that the back batter is on the same side of the vertical as the face batter. Isosceles walls are indicated by the equality of the two batters.

Conditions. – Angle $\phi = 33° \; 41'$, or $1\frac{1}{2}$ to 1. Weight w of earth = 100 lbs. per cubic foot. Weight W of masonry, etc., = 167 or 125 lbs., as stated. Calculated by the general formula in Appendix (IV).

Numerical Values for Walls of Equilibrium.

Material of wall.	$\frac{w}{W}$	Tan δ Face batter.	Tan θ Back batter.	Ratio of base of wall to height.
Masonry.	$\frac{100}{167} = \frac{3}{5}$	Vertical	1.1300	1.1300
		$\frac{1}{12}$ = 0.0833	0.9123	0.9956
		$\frac{1}{4}$ = 0.2500	0.4885	0.7385
		0.3191	0.3191	0.6382
		0.4558	Vertical	0.4558
Dry Masonry.	$\frac{100}{125} = \frac{4}{5}$	0.3810	0.3810	0.7620
		$\frac{1}{2}$ = 0.5000	0.0665	0.5665
		0.5263	Vertical	0.5263
Rip-Rap.	$\frac{100}{125} = \frac{4}{5}$	$\frac{1}{1}$ = 1.0000	– 0.9240	0.0760
		0.8366	$-\frac{2}{3}$ = 0.66	0.1700

The above table may be extended by means of the general formula to include any class of loose material, supported by walls of any given weight per cubic foot. The formulæ for walls with either the face or the back vertical, deduced from the general formula, are also given in the Appendix. If preferred, the forms of such walls of equilibrium may be found from the original formula $T = \frac{1}{2} w p^2$ and the moment of stability, by means of direct trial and successive approximations.

We may also obtain a comparison of the pressures exerted by different classes of loose material, by means of a ratio between them. Let the two classes be defined as follows :—

T_1 = horizontal thrust ; w_1 = weight; ϕ_1 = angle of repose.

T_2 = " " w_2 = " ϕ_2 = " "

Then $$\frac{T_2}{T_1} = \frac{w_2 \cos^2 \phi_2}{w_1 \cos^2 \phi_1}$$

Numerical examples. $\phi_1 = 33° \ 41'$ for sand, and $\phi_2 = 45°$ for loose rock. Then :—

$$\frac{T_2}{T_1} = \frac{w_2}{w_1} \frac{13}{18}$$

Also, if the stone weighs 167 lbs. per cubic foot, the broken stone or loose rock formed from it will have 40 per cent. of void, and will weigh 100 lbs. per cubic foot, which is the same as for sand. Hence $w_2 = w_1$, and we have

$$T_2 = 0{\cdot}722 \ T_1$$

These ratios, whether algebraical or numerical, will be closely approximate for any considerable height of surcharge.

Grade Walls, or walls supporting loose material limited by a horizontal surface at the level of the crest of the wall.

Here we have the loose material limited by an arbitrary surface and the element of friction between it and the wall is no longer eliminated as in the case of a surface at repose. Hence there are two conditions or assumptions to be clearly distinguished :—

(1) The existence of friction between the loose material and the back of the wall, equivalent in amount to the angle of friction ψ. In this case the best solution is the graphical one, as shown in Figure 15 in the Appendix, the surface line being made horizontal. The algebraical method is complicated ; although some simplification is obtained by assuming $\psi = \phi$, the angle of friction for the material itself, which is usually a close approximation.

(2) No friction between the loose material and the wall. This is clearly an assumption ; but it is generally employed in practice, because it gives a greater value for the thrust than when the friction is taken into account, and therefore leaves a greater margin on the side of

stability. This additional stability may also be taken to make up for the effect of vibration occasioned by either a train or waggon where the wall supports a road-bed.

The graphical and algebraical methods for this case are given in the Appendix (III), and also the proof of the following formula for the horizontal thrust T: –

$$T = \frac{wh^2}{2} \cdot \frac{1 - \sin(\phi - \epsilon)}{1 + \sin(\phi + \epsilon)}$$

(The sign of ϵ will be reversed if the inclination ϵ is on the other side of the vertical. See Fig. 20.)

We can obtain an interesting comparison between the amount of pressure occasioned by the same kind of material in this case and in the original case of an unlimited slope at repose, if we simplify the formulæ to represent a wall with the back vertical.

Let T_1 = horizontal thrust for an unlimited slope at the angle of repose ϕ, as already found.

T_2 = horizontal thrust for a horizontal surface.

h = vertical height of the wall.

$$T_1 = \tfrac{1}{2}\, w\, p^2 = \tfrac{1}{2}\, w\, h^2 \cos^2 \phi$$

$$T_2 = \frac{w\,h^2}{2} \cdot \frac{1 - \sin\phi}{1 + \sin\phi} = \frac{w\,h^2}{2} \cdot \tan^2 \tfrac{1}{2}(90 - \phi)$$

$$\frac{T_2}{T_1} = \frac{\tan^2 \frac{1}{2}(90^\circ - \phi)}{\cos^2 \phi} = \frac{1}{(1 + \sin\phi)^2}$$

The following numerical values from this formula will serve as illustrations of the ratios obtained :—

For	$\phi = 30^\circ\ 00'$	$T_2 = 0.4444\ T_1$
"	$\phi = 33^\circ\ 41'$	$T_2 = 0.4137\ T_1$
"	$\phi = 38^\circ\ 39'$	$T_2 = 0.3788\ T_1$
"	$\phi = 45^\circ\ 00'$	$T_2 = 0.3431\ T_1$

From such ratios we can obtain a decided advantage in simplicity of method ; for we may first determine the best form of wall for an unlimited slope of the class of loose material in question, and then find its factor of safety when the material is cut down to a horizontal surface. If this is still insufficient, an additional amount can be added to the cross section to bring the safety up to any required factor.

Walls of Equilibrium and actual Cross-Sections.—After determining the wall of equilibrium ABC for any conditions, by the methods described, an addition can readily be made to the cross-section to give the wall any desired stability, as shown in the shaded portions of the figures. As the pressure against the wall is unchanged by this addition, the factor of safety will be the ratio of the moment of stability of the completed wall to the moment of stability of the wall of equilibrium. An advantage in this method is also that the actual margin of safety is apparent.

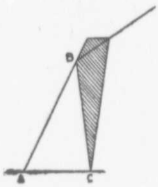


Fig. 7

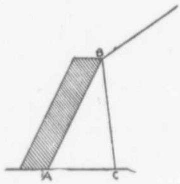


Fig. 8

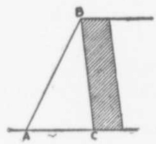


Fig. 9

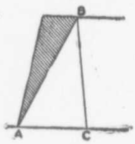


Fig. 10

Loose Rock supporting Earth.—Failure of the supporting mass by sliding at an angle, as described at first as mode (C).

The mode of failure assumed so far has been that the wall will turn around the outer edge of the base, and fail by upsetting. This implies that the foundation is unyielding, and also that the friction on the base is sufficient to prevent the wall from pushing forward horizontally. These modes of failure have been explained at the outset and indicated by the letters (A) and (B). We will now refer to the third mode of failure, in which the supporting mass may give way by sliding at an angle on itself. This may occur when rip-rap or loose rock is used for the support of earthwork. In the region referred to, there were two problems of this kind. (1) A side-hill embankment made of loose rock on the outside. (2) A cutting with its slope supported by rip-rap.

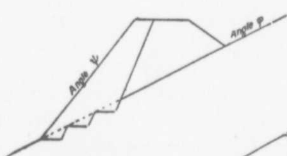


Fig. 11

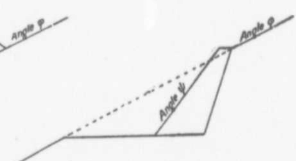


Fig. 12

These conditions are illustrated by the figures, and the question is the thickness of rock embankment or rip-rap required. As the supporting material is itself loose, the support it gives will be due to its weight and friction only ; and it will fail by shearing or sliding along some line of least resistance. The position of this line in the mass involves the determination of a minimum under conditions which may be thus stated :—

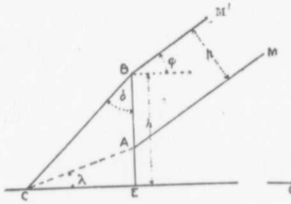


Fig 13

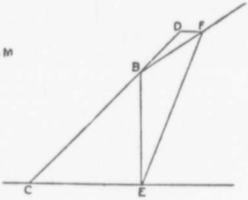


Fig. 14

Let CBE be the mass which supports loose material with a surface BM′ at the angle of repose. The line of least resistance CA, or line of shear along which the mass will give way by sliding, will be inclined at some angle λ to the horizon. Required the value of λ which will give a minimum value to this resistance ; and the angle δ which will give sufficient thickness to insure stability. It is to be noted that the thickness p of the layer supported varies also with the angle λ. The simple expression already found for the horizontal thrust T of this layer enables a relation between the angles δ, λ, and the angles of friction ϕ and ψ to be found, as given in the Appendix (V).

For the most important practical case of loose rock supporting sand and gravel, the weight of the two kinds of material per cubic foot is practically the same ; and the stability depends entirely on the increased angle of friction in the loose rock, the angles being $\phi = 33^\circ\,41'$ and $\psi = 45^\circ$ respectively. If we make also $\delta = 45$ to here will be equilibrium in the lower part of the mass CBE up to the inclination $\lambda = \phi$; but above this the top of the triangle will be pushed off by the pressure. There will be a value of λ which gives a minimum, however, if δ is greater than 45°, and equilibrium will then obtain. In practice, the upper part of the triangle can be further strengthened by adding the area BDFE.

For the support of a horizontal surface, as in the case of embankment Fig. 11, this solution is sufficiently close, as the relatively low cost of loose rock, even when it has to be obtained by excavation, makes it allowable to use an amount considerably in excess of theoretical requirements. It also makes an embankment practicable on steep side-hill ground, and where a masonry wall would be much more expensive.

In the case of a cutting, Fig. 12, the reverse is usually the case, because the volume of masonry required is so much less than the amount of loose rock laid on as rip-rap, that at ordinary prices the masonry is the more economical of the two. This is probably the reason that this problem has not received more attention; although there are many cases in which it is of practical value.

APPENDIX.

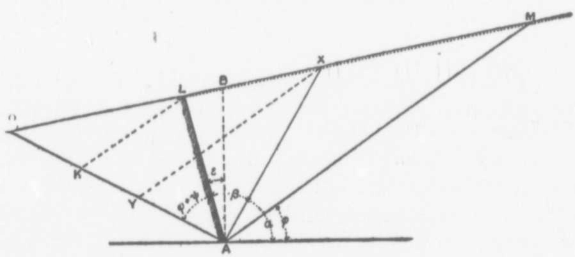


Fig. 15

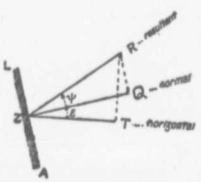


Fig. 16

Graphical method of determining the thrust of loose material limited by any plane LM, against the back of a wall AL.

Let ϕ = angle of repose; *i.e.* the angle of friction of the loose material on itself.

ψ = angle of friction of the loose material against the wall.

ϵ = inclination of back of wall to the vertical.

w = weight of loose material per unit of volume.

Q = pressure normal to the wall.

R = resultant pressure at an angle ψ to the wall.

T = horizontal component of R.

The relation of Q, R and T is shown in the lower figure.

I. *General Method.*—Applicable to all inclinations of the surface and of the wall, and to all classes of loose material.

Draw AM at an angle ϕ with the horizontal.

Draw AO at an angle $\phi + \psi$ with AL.

Draw LK parallel to AM; and take OY a mean proportional between OK and OA; and draw YX parallel to AM. Then XA will be the line limiting the prism of maximum pressure.

Also the general equation giving the value of the pressure is:—

$$\frac{Q}{\cos\psi} = \frac{w}{2} \cdot \sin(\alpha + \beta \quad \psi)\,\overline{\text{AY}}^2$$

$$\text{or } \frac{Q}{\cos\psi} = \frac{w}{2} \cdot \cos(\psi + \epsilon)\,\overline{\text{AY}}^2$$

and also since $R \cos\psi = Q$ and $T = R\cos(\psi + \epsilon)$

$$\text{we have } T = \frac{w}{2} \cdot \cos^2(\psi + \epsilon)\,\overline{\text{AY}}^2$$

(If ϵ is on the other side of the vertical, its sign will be negative.)

For proof of this method, and the resulting general equation, see Collignon, "*Mécanique Appliquée,*" Book VIII, chap. IV.

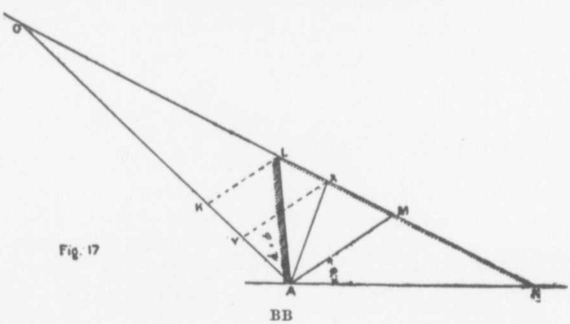


Fig. 17

BB

To show how general this method is, the following additional examples may be given. In the second case the point J is found by making LJ parallel to AH, in order that the triangles ALH and AJH may be equal in area, for compensation.

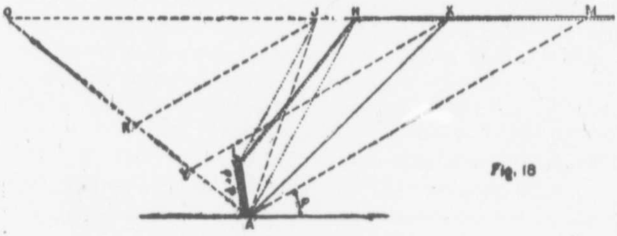
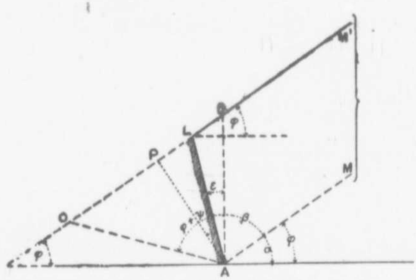


Fig. 18

II. *For an unlimited surface at the angle of repose.*



In this case the following simplifications take place. As LM′ is at the angle of repose, and therefore parallel to AM, the points K and Y coincide with O; and the length AY becomes AO. Also, since AX coincides with AM, the angle a becomes the angle ϕ; and the equation becomes:—

$$\frac{Q}{\cos\psi} = \frac{w}{2} \cdot \sin(\beta + \phi + \psi)\ \overline{OA}^2$$

$$\text{or}\quad \frac{Q}{\cos\psi} = \frac{w}{2} \cdot \cos(\psi + \epsilon)\ \overline{OA}^2$$

$$\text{and } T = \frac{w}{2} \cdot \cos^2(\psi + \epsilon)\ \overline{OA}^2$$

Let $p =$ the perpendicular distance AP; then as

$$p = \sin\ (\beta + \phi + \psi)\ OA = \cos\ (\psi + \epsilon)\ OA,$$

we have finally, $T = \frac{1}{2}\ w\ p^2$

NOTE.—In Rankine's "Civil Engineering," a formula corresponding to this is given for the special case of a wall with a vertical back. In Article 183, IV., the pressure parallel to the surface is denoted by P' and for a surface sloping at the angle of repose he gives (in our notation) $P' = \frac{1}{2}\ w\ \cos\ \phi\ \overline{AB}^2$. As $P'\cos\phi = T$, and AB $\cos\phi = p$, the formulæ are evidently identical.

III. *Surface horizontal; as for Grade Walls.*

Condition; no friction between loose material and wall. With this condition, Prôny has shown for a vertical wall, and Français for an inclined wall, that the line AX, limiting the prism of maximum pressure, will bisect the angle LAM.

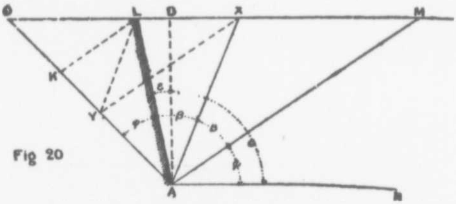


Fig 20

As the general method is also applicable to a horizontal surface, we obtain the formulæ at once by introducing the new conditions. Using the same notation as before, and making the vertical height AB $= h$, we have in this case $\psi = o$ and the normal pressure Q therefore coincides with the resultant R.

$$\text{Hence } R = Q = \frac{w}{2}\cos\ \epsilon\ \overline{AY}^2$$

and since we now have $T = R\cos\epsilon$

$$T = \frac{w}{2}\cos^2\ \epsilon\ \overline{AY}^2$$

The value of AY can be found, and the equation reduced to the algebraic form thus:—

Since AX bisects LAM, it can be shown that $OY = OL$; and therefore in the triangle OAL:—

$$AY.\cos\epsilon = OA\,(\cos\epsilon - \sin\phi)$$

$$\text{Also } OA.\cos(\phi+\epsilon) = AB = h$$

$$\text{Hence } T = \frac{wh^2}{2}\cdot\frac{(\cos\epsilon - \sin\phi)^2}{\cos^2(\phi+\epsilon)}$$

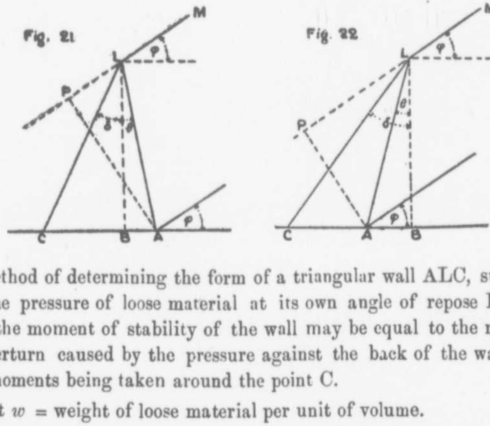
From the evident relations between the angles this becomes

$$T = \frac{wh^2}{2}\cdot\frac{1-\sin(\phi-\epsilon)}{1+\sin(\phi+\epsilon)}$$

(If ϵ is on the other side of the vertical AB, its sign in the above equation will become reversed.)

NOTE.—In the special case of a wall with a vertical back $\epsilon = o$, and this formula then becomes identical with Rankine's. See "Civil Engineering," Article 183, IV.

IV. *Walls of Equilibrium, general formula.*



Method of determining the form of a triangular wall ALC, supporting the pressure of loose material at its own angle of repose LM, so that the moment of stability of the wall may be equal to the moment of overturn caused by the pressure against the back of the wall AL, the moments being taken around the point C.

Let w = weight of loose material per unit of volume.

W = weight of material of the wall per unit of volume.

ϕ = angle of repose of the loose material.

δ = angle of the face batter; and $tan\ \delta = n$.

θ = angle of the back batter.

h = vertical height LB.

T = horizontal component of the pressure.

In solving the problem, either the back batter θ or the face batter δ must be assumed, and the other calculated from it.

The weight of the wall made up by the sum or difference of the two triangles CLB and ALB is :—

$$\tfrac{1}{2}\ Wh^2 \tan\delta \pm \tfrac{1}{2}\ Wh^2 \tan\theta$$

The moment of stability around the point C is :—

$$\frac{Wh^3}{6}\ (2\ n^2 + 3\ n \tan\theta + \tan^2\theta)$$

The moment of overturn around C is :—

$$\frac{h}{3}\cdot\ T = \frac{wh^3}{6}\ (\cos\phi + \sin\phi\tan\theta)^2$$

By equating these moments, the two following forms are obtained according as the equation is arranged to solve for n or for $tan\ \theta$;

$$2\ n^2 \pm 3 \tan\theta.\ n + \tan^2\theta = \frac{w}{W}\cdot(\cos\phi \pm \sin\phi\tan\theta)^2$$

$$\text{or}\ \left(1 - \frac{w}{W}\sin^2\phi\right)\ \tan^2\theta \pm \left(3\ n - \frac{w}{W}\cdot\ 2\sin\phi\ \cos\phi\right)\tan\theta$$

$$+ \left(2\ n^2 - \frac{w}{W}\cos^2\phi\ \right) = o$$

(The negative sign must be used when θ is on the left of the vertical in the figure; and in this case also the negative value of the square root must be taken in solving the equation for tan θ)

When the back batter is vertical, $\theta = o$

$$\text{and}\ n^2\ \text{or}\ \tan^2\delta\ = \tfrac{1}{2}\cdot\frac{w}{W}\cdot\ \cos^2\phi$$

When the face batter is vertical, $\delta = o$, *i.e.* $n = o$ and

$$\left(1 - \frac{w}{W}\sin^2\phi\right)\ \tan^2\theta. - \left(\frac{w}{W}\cdot\ 2\sin\phi\ \cos\phi\right)\ \tan\theta$$

$$- \ \frac{w}{W}\cos^2\phi = o$$

V. *Loose Rock supporting Earth*, or other loose material. (See Figure 13.)

A relation between the angles in this case can best be obtained by finding a ratio between the pressure of the layer p and the frictional support afforded by the mass ABC along the line AC. The maximum value of this ratio will correspond with the line of minimum stability.

Let Q = weight of the triangle ABC.

T = horizontal thrust of the layer p against AB.

ϕ = angle of repose of the loose material.

w = weight per unit of volume of the loose material.

ψ = angle of friction of the supporting material ABC.

W = weight per unit of volume, of the above.

Then as the back AB is vertical,

$$Q = \tfrac{1}{2}\, Wh^2\, (1 - \tan\delta \tan\lambda) \tan\delta$$

Inclined friction along AC $= Q\,(\tan\psi - \tan\lambda)\cos\lambda$

Hor'l. component of this friction $= Q\,(\tan\psi - \tan\lambda)\cos^2\lambda$

Hor'l. thrust of layer $p = \tfrac{1}{2}\, w\ p$

$$= \tfrac{1}{2}\, w\, h^2\, (1 - \tan\delta\tan\lambda)_2 \cos^2\phi$$

Hence for ratio of hor'l. thrust to hor'l. component of friction :—

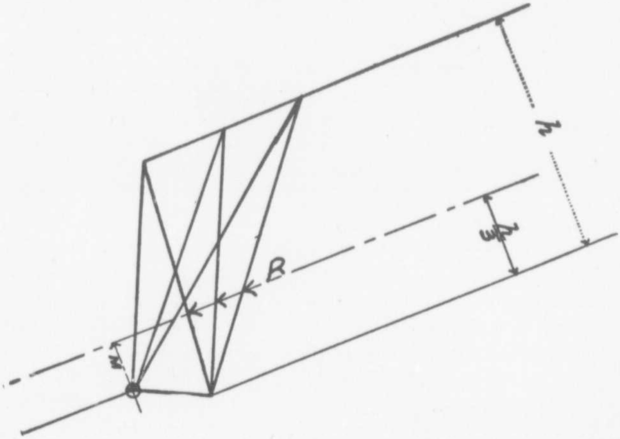
$$\frac{\text{Thrust}}{\text{Friction}} = \frac{w}{W} \cdot \frac{\cot\delta - \tan\lambda}{\tan\psi - \tan\lambda} \cdot \frac{\cos^2\phi}{\cos^2\lambda}$$

For stability this ratio must not be less than unity; *i.e.* the angle δ must be sufficiently large to prevent the ratio from falling below unity for any value of λ between $\lambda = o$ and $\lambda = 90^\circ - \delta$.

DISCUSSION.

Mr. Allison said :—The author is to be congratulated on having given us a new method of treating an old problem. There are a few points, however, which would appear to require further consideration. Mr. J. L. Allison

In section A of the eighth paragraph, it is stated that for a surface with any elasticity, the resultant pressure against the base of a wall on the point of overturning lies at the forward limit of the middle third. Is it necessary to state anything further than that on rock the resultant will closely approach the forward edge, and as the material becomes more yielding it will recede from it? The author's contention, that the toe of a wall should seldom be considered as the point about which rotation would tend to take place, is well taken.



Attention has already been drawn by Mr. Irwin to the fact, that only the horizontal component is considered in calculating the overturning moment. Whenever there is a vertical component, it must be either helping or hindering the horizontal, and should be considered.

Figures 2, 3 and 4 show different moment arms for the same thickness of layer supported by different walls. Rankine gives the point of application of the resultant pressure on the back of the wall, the same as shown in the figures, and its direction as parellel to the surface. This would give the same moment arm M for the three walls shown in the cut above.

Prof. C. B. Smith.

Mr. Smith said:—The unfortunate part of any paper on retaining wall design, attacked entirely from a theoretical standpoint, is indicated in that paragraph in Mr. Dawson's paper where the author determines a triangle of supposed safety based on several assumptions not positively known, and then adds a large block of masonry for safety, thereby admitting that theorists have not yet been able to design a wall to retain earth under varying conditions, not determinable; this is not disparagement of theory, but merely an admission that unless sufficient premises are given, a solution is only approximate, and must call experience and trial to its aid. The speaker remembers having occasion to compile the results of tests on models of retaining walls, and the opinions of engineers regarding retaining walls in general according to the best evidence at that time (1883), and he found several instances where walls, theoretically safe, had fallen, and others, which, according to theory, should have fallen that time had shown to be perfectly safe.

The conclusion seemed to be, that although theory could not design a wall as well yet as experience, still, where experience was lacking (under new conditions), theory was very valuable in proportioning a wall with a factor of safety somewhat similar to other walls which were found perfectly safe. Mr. Irwin's remarks also pointed to this as being the chief use, so far, of a theoretical consideration of retaining walls.

Mr. H. E. Vautelet.

Mr. Vautelet said he had read with much interest the valuable paper written by Mr. Dawson, and would like to present a few observations.

1. The analogy between the formula for the special case of an infinite slope with the formula for the hydrostatic pressure of a liquid is a well-known fact, and the horizontal component not only for this case, but for all cases of loading, can evidently be put under the same form; for taking Mr. Dawson's notation

$$R = \frac{w}{2} \frac{X^2}{\sin(\alpha + \beta + \psi)}$$

and as $T = R \sin(\alpha + \beta + \psi)$

it follows that in every case $T = \dfrac{w \quad X^2}{2}$

2. Mr. Dawson says: "In the case of rectangular wall for an infinite surcharge, in the series recommended by Poncelet, the base is 0.934 of the height, while the stability is only 1.86, which shows at once how far such a section is from being economical."

Let us admit for a moment that the walls A B^1 C^1 to A B^5 C^5 given by Mr. Dawson are really wall of equilibrium, and also that any two different walls, such as the ones shewn in Fig. 10, can be compared by the relation between their moments of stability.

Considering Mr. Dawson's cheapest wall of equilibrium A B^5 C^5, let us find how much masonry must be added as per Fig. 10, the front batter being too large, for practical reasons, to give it a stability of 1.86.

We shall build the wall vertical in front.

The base is as given $h \times 0.4558$, h being the height B^5 C^5

Let us call h^1 the height of the new wall.

We have:

$$\frac{h^1 \times h^2 \times \overline{0.4558}^2}{2} = 1.86 \times \frac{h}{2} \times \frac{2\,h^2 \times \overline{0.4558}^2}{3}$$

$$\frac{h^1}{2} = 1.86\ \frac{h}{3} \qquad h^1 = 1.24\ h$$

and the cross-section is $1.24\ h \times 0.4558\ h = 0.565\ h^2$

Let us now see how Poncelet's wall compares with the wall A B^5 C^5 strengthened so as to give it the same stability.

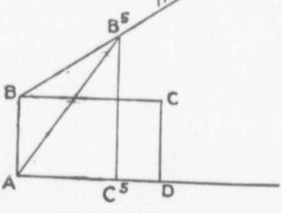


Fig 1

A B C D is a section of Poncelet's wall mentioned by Mr. Dawson.

$B^5\ C^5 = h$

$A\ C^5 = 0.4558\ h$

$A\ B = h - \frac{2}{3} \times 0.4558\ h = 0.6962\ h.$

$A\ D = 0.934 \times 0.6962\ h = 0.65\ h.$

Section A B C D $= 0.6962\ h \times 0.65\ h = 0.45\ h^2$

Poncelet's wall for the same stability is therefore more economical than Mr. Dawson's.

3. The relative amounts of masonry in different walls are a resultant of theory and practice.

It is a well known fact that, in theory, when a berm extends for the whole width of the top, a wall with a vertical back has less masonry than any other; but in practice, if we take into account the amount of foundation and excavation and the cheapness of work, the writer thinks that, more especially with a surcharge, a rectangular wall with a small berm, as proposed by Poncelet, will be found cheaper than any; and that where there is no surcharge, and where the effects of frost have to be taken into consideration, a modification of the rectangular wall will be, and in fact is, used by every engineer.

For instance, if we compare Poncelet's wall with the C. P. Ry's wall, we find that up to 25′ Poncelet's has less masonry, and for larger heights, leaving the foundations out, the C. P. Ry's wall has less masonry, and that the stability of both is about the same.

4. Let us now consider the "table of numerical values for walls of equilibrium."

The values are found by the general formula given in Appendix IV.

But to establish the formula, Mr. Dawson leaves out altogether the vertical component of the resultant R.

Now if for the sake of simplicity we make $\psi = 0$ (Mr. Dawson says that it is an assumption employed in practice), and if we admit that the back batter of the wall A B_1 C_1, Fig. 5, is 1 instead of 1.13, the wall A B_5 C_5 is really a wall of equilibrium, but in A B_1 C_1 the vertical component V is equal to the horizontal component T, and the moment of overturn is — $\frac{Th}{3}$ and the wall A B_1 C_1 has a stability of 3 against a co-efficient of 1 for the wall A B^5 C^5.

In fact, the resultant of A B_1 C_1, if $\psi = 0$, strikes between the outer third and the middle of the base, and if $\psi = \phi$ (as usually taken) it strikes between the center of the inner third, and the wall has really an unnecessary amount of stability.

Whereas in the first case ($\psi = 0$) the wall A B^5 C^5 is a wall of equilibrium, and in the second case ($\psi = \phi$) is just about right.

5. It should be added that it is difficult to compare directly walls, like the ones shewn in Figs. 7 and 10. The writer believes that the walls given in the series recommended by Poncelet are obtained by finding, first, a rectangular wall of equilibrium, and taking for the wall as recommended a base 1.8 times larger. The total resultant falls then in the middle third, and the wall has ample stability.

But we cannot compare any two walls by the relation between their moments of stability. The stability is measured by the distance to the toe of the resultant of the total thrust R and of the weight of the wall and by the angle it makes with the courses of the stones.

For instance, if we compare the wall A B C and the rectangular wall having the same base and a height = 1.24 times its base, we find that the resultant in both cases strikes the base at very nearly the same point, and that the only improvement obtained by the large addition of masonry is that the resultant strikes the base more nearly perpendicularly.

To resume : the different triangular walls compared by Mr. Dawson are not walls of equilibrium and are not comparable to each other.

Even if they were comparable to each other, the forms derived in Figs. 7 and 10 could not be directly compared to the original walls.

6. The writer would also like to mention the fact that a great many methods and formulæ proposed for the solution of this problem are derived from the theory of Coulomb (1773), completed by Poncelet (1840), both of them following in the footsteps of Vauban's, and that the results under different forms are identical.

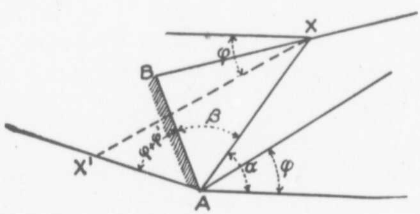
As an instance take one of the forms of Weyrauch's formula

$$\frac{E = \cos^2(\phi + \omega)}{\cos(a + \delta)} \times \frac{K^2 \gamma}{2}$$

and Poncelet's

$$\frac{Q}{\cos \phi^1} = \frac{\pi}{2} \sin(a + \beta + \varphi^1) \times \frac{2}{A X^1}$$

as given by Collignon, in which the notations are as follows:



A X = K

Fig. 2

	Weyrauch.	Poncelet.
Total thrust of earth against wall AB	E	$\frac{Q}{\cos \phi'}$
Angle of repose of earth	ϕ	ϕ
Angle made by surface of rupture AX with vertical	ω	$90°—a$
Weight of unit of earth	γ	π
Angle made with normal to BA by E	δ	ϕ'
Angle made by rear face of wall with vertical	a	$a + \beta—90°$

from triangle AXX′ we find

$$K = AX' \frac{\cos [(a+\beta-90°) + \phi']}{\cos [(90°-a)+\phi]}$$

replacing in Poncelet's formula

$$\frac{Q}{\cos \phi'} = \frac{\pi}{2} \times \frac{\cos^2 [(90° - a) + \phi]}{\cos [(a + \beta - 90°) + \phi']} \times K^2$$

and replacing by Weyrauch's notation

$$E = \frac{K^2 \gamma}{2} \times \frac{\cos^2 (\phi + \omega)}{\cos (a + \delta)}$$

which shows the identity of the two formulæ.

Dr. M. Murphy. Dr. Murphy said the paper proposes a new method for the design of retaining walls under somewhat unusual conditions, taking as a basis the construction of a wall to support an unlimited length of slope of loose material at its own angle of repose, and to bring all other cases into relation thereto. The author takes the natural surface in equilibrium as his data, instead of an arbitrary limiting surface of support usually taken by other authorities, and this brings the question more into relation with the hydrostatic problem, for walls to support water.

Sand, earth or *coarse gravel*, etc., such as we have to deal with in every day practice, may be considered as *semi-fluids.* They are fluids so far as to require external support, the adhesion of their parts is of course greater than water, which is in equilibrium only when its surface is level; but the *disintegrated masses*, or semi-fluids in question, may be at rest, though their surface be inclined.

We can compare the pressure of a disintegrated or semi-fluid mass with that of water. In the latter the pressure is $\frac{1}{2}h^2 V_1$ for unit breadth when h = height under pressure. In the case of earth, on the other hand, we have the pressure

$$P=\frac{1}{2} h^2 e r_1 \left[\text{tang.} \left(45°-\frac{\rho}{2}\right) \right]^2$$

when r_1=density of water, e the specific gravity of the semi-fluid and ρ=angle of repose. Hence the pressure of the earth is always

$e\left[\text{tang.}\left(45^\circ-\frac{\rho}{2}\right)\right]^2$ times as great as the pressure of water, or the pressure of a semi-fluid may be set as equal to a perfect fluid of pseciflc gravity $e\left[\text{tang.}\left(45^\circ-\frac{\rho}{2}\right)\right]^2$

The pressure of earth increases gradually from the surface downwards, or is proportional to the pressure of height.

It can be shown likewise that the centre of pressure of earthwork, etc., coincides with the centre of pressure of water, and that where the surface is a rectangle it is $\frac{1}{3}$ of the height h from the base.

If we take any natural surface of slope, the surface and the ground beneath it, for a certain depth into which we require to penetrate, to have a definite and uniform arrangement of parts, these parts or particles to balance each other, just self-sustaining, yet to be in equilibrium, to be at rest, then, for our road or railway, we have to cut horizontally into the face of this inclined slope, and to resist the thrust from above for the depth of the cut we have made by a retaining wall sufficiently stable to sustain the slope or prism we have damaged by our cut, it is evident that the strength of the wall must be equivalent in sustaining power to the earth or loose material before removal and the pressure against the wall, the horizontal component or horizontal thrust may be deduced from these considerations.

The author has, in the writer's opinion, done good service in bringing forward the subject so plainly and so clearly, and also for pointing out the tendencies for failures by overturning, etc.; for when it is known where a bad tendency is likely to exist, however slight it might be, practical precautions can be taken to prevent it. But it seems to him the subject should be carried further, so as to embrace the measure of the effects of these tendencies. If the author would follow up the subject in the same concise and able manner in which he has commenced, by giving the Society the pressures that are likely to be brought to bear against surcharged retaining walls under the different stresses that must ensue by reason of the extremes of temperature, the communication would be more complete, and by that means theory and practice would be thoroughly reconciled.

With regard to the general conceived opinion, that the theory and practice are at variance with each other, in relation to the stability of retaining walls, the writer believes it to be a truism; that theory and practice can never be at variance, when theory itself is perfect and sufficient, so as to embrace all points, and when the practice is carefully observed and considered.

Mr. H. Irwin. Mr. Irwin :—There has been so much already written on the theory of retaining walls that one might suppose that there was little left that was new to be said on the subject, and yet Mr. Dawson's method of treating the case of loose rock supporting earth was quite new, to him at any rate, and seems to indicate, in general terms, the proper method of dealing with that case.

He did not, however, agree with Mr. Dawson as to the details of his method, especially where he virtually states that the "simple expression, for the horizontal thrust," is all the thrust of the earth that has to be dealt with, ignoring altogether the vertical component of the thrust, which he seems to think proper (though his reason is not clearly given), because he considers that the friction between the earth and the back of the wall should not be considered. Now if the back of the wall were perfectly smooth, and if the pressure were applied, by means of a rigid rod, at the centre of pressure, then the pressure normal to the wall, and no other, would have to be dealt with; but in the present case the pressure is applied by a confined mass of earth, so that its total pressure, determined both in direction and amount by the friction between its own particles, must be considered as applied to the wall in its entirety; and the friction between the earth and the back of the wall really only comes into play, towards keeping the wall from overturning, after motion has begun, and even then, by considering the point round which a wall will begin to rotate, it will be seen that in the case of a wall with a vertical or nearly vertical back, this friction would be of little use towards keeping the wall from overturning, as the wall, in rotating, would be falling away from the earth and diminishing its pressure and its frictional effect.

As to the statement that "the horizontal component of the pressure "on the wall is independent of the existence or non-existence of fric- "tion between the loose material and the wall"—it does not seem clear why the *horizontal* component should be referred to in this connection, except in the case of a wall with a vertical back. If the existence or non-existence of said friction affected the earth pressure, surely it would be the component normal to the back of the wall that would be independent of the friction; but from what has been already said it would appear that said friction does not really enter into the case until motion of the wall commences.

In fact Mr. Dawson's idea that walls can be designed by dealing with the horizontal component alone seems to be entirely erroneous.

It will be noticed in Appendix 5 to Mr. Dawson's paper, dealing with the case of loose rock supporting earth, that he finds the ratio between the thrust and the friction which resists it, by comparing the horizontal component of the thrust with the horizontal component of the friction of the mass of the dry wall which may be just about to slide along the plane of least resistance.

It would seem, however, that the proper method of dealing with the case is to compare the friction of the portion of the wall, which may be just about to slide, along the plane of fracture, and also the friction produced by the component of the earth pressure normal to the plane of fracture, with the thrust of the component of the earth pressure parallel to the plane of fracture.

Now the earth thrust, in the case considered in Appendix 5, where the back of the wall is vertical, is $\frac{w\ p^2}{2 \cos \phi}$, and the component of this, parallel to the supposed plane of fracture, is $\frac{w\ p^2}{2 \cos \phi} \times \cos (\phi - \lambda)$, ϕ being supposed to be greater that λ, as in figure 13 of Mr. Dawson's paper; but $\frac{w\ p^2}{2 \cos \phi} = \frac{1}{2} w\ h^2 (1 - \tan \delta \tan \lambda)^2 \cos \phi$, therefore component of earth pressure along plane of fracture $= \frac{1}{2} w\ h^2 (1 - \tan \delta \tan \lambda)^2 \cos \phi \cos (\phi - \lambda)$; also the component of the earth thrust normal to the plane of fracture $= \frac{1}{2} w\ h^2 (1 - \tan \delta \tan \lambda)^2 \cos \phi \sin (\phi - \lambda)$; and the friction produced by this along the plane of fracture is $= \frac{1}{2} w\ h^2 (1 - \tan \delta \tan \lambda)^2 \cos \phi \sin (\phi - \lambda) \tan \psi$, also the friction due to the portion of the wall marked A B C is $= Q (\tan \psi - \tan \lambda) \cos \lambda = \frac{1}{2} W h^2 (1 - \tan \delta \tan \lambda) \tan \delta (\tan \psi - \tan \lambda) \cos \lambda$ therefore $\frac{\text{Thrust}}{\text{Friction}} =$

$$\frac{\frac{1}{2} w\ h^2 (1 - \tan \delta \tan \lambda)^2 \cos \phi \cos (\phi - \lambda)}{\frac{1}{2} w h^2 (1 - \tan \delta \tan \lambda)^2 \cos \phi \sin (\phi - \lambda) \tan \psi + \frac{1}{2} W h^2 (1 - \tan \delta \tan \lambda)}$$

$$\times \frac{1}{\tan \delta (\tan \psi - \tan \lambda) \cos \lambda} =$$

$$\frac{w (1 - \tan \delta \tan \lambda) \cos \phi \cos (\phi - \lambda)}{w (1 - \tan \delta \tan \lambda) \cos \phi \sin (\phi - \lambda) \tan \psi + W (\tan \psi - \tan \lambda) \cos \lambda \tan \delta}$$

from which we get $\frac{\text{Thrust}}{\text{Friction}} =$

$$\frac{w (\cot \delta - \tan \lambda) \cos \phi \cos (\phi - \lambda)}{w (\cot \delta - \tan \lambda) \cos \phi \sin (\phi - \lambda) \tan \psi + W (\tan \psi - \tan \lambda) \cos \lambda}$$

which is a somewhat similar expression to that deduced by Mr. Dawson, but, of course, much longer.

This expression, however, does not fix the position of the plane of fracture in the wall, so that it seems that trial would have to be made of several planes at different heights.

It may be remarked also that, except in the case of very small stone, or of an immense mass of larger stone, the plane of fracture would depend largely on the arrangement of the stones in the wall, so that any formulæ must be considered as only approximate, with a tendency to err on the safe side.

In the case of dry walls, depending on the internal friction of the mass of the wall, it would seem that it would be advisable not to make the back slope backwards too much, because in that case the thrust of the earth tends to exert a lift against the back of the wall.

Mr. Dawson's statement that authors have "*usually* considered the "case of some arbitrary limiting surface for the earth" seems rather overdrawn.

As regards comparison with water pressure, any of the formulæ for the ordinary cases of earth pressure can be easily compared with those arising from water pressure.

In dealing with a wall of "*considerable surcharge*," it would seem rather to savour of guess work to "first design a wall of equilibrium to "support an unlimited surface at the angle of repose," and "then to "allow the difference between that and the actual surcharge as a margin "of safety, and if necessary to increase the wall itself for greater sta- "bility," because after changing the section of the wall or taking a limited for an unlimited surface of repose, the effect of the change would not be known without making a fresh set of calculations or a new diagram.

Surely it would seem shorter and more reasonable to assume a section of wall in accordance with some such well-known rules as Trautwine's, and then to make the necessary diagram to ascertain how the resultant pressure cuts the wall.

Mr. W. B. Dawson.

Mr. Dawson:—The author desires to thank those who have taken part in the discussion for their careful consideration of the question and the criticisms they have brought to bear upon it.

Some exception has been taken to the horizontal component of the pressure against the wall being alone considered. The advantage of dealing with this component only has been pointed out in the paper; but it may be well to add that the actual or resultant pressure is readily found from this horizontal component. The angle between the horizontal component and the resultant depends upon the back batter of the wall, and the angle of friction, if the friction is taken into account. The resultant pressure in both magnitude and direction may thus be accurately determined from the horizontal thrust.

After designing a wall of equilibrium, it is clear that some additional material must be added to give it stability. This, however, can be done readily and definitely, as it is only necessary to ascertain by what percentage the moment of stability is increased by the area added to the cross-section of the wall ; and this method avoids the need of recommencing the whole calculations for the altered form of the wall.

In the case of a wall of " considerable surcharge," referred to by Mr. Irwin, it is to be noted that the upper part of the surcharge has relatively little effect in increasing the pressure on the wall if the surcharge is already considerable. For example, with a surcharge extending from the top of the wall at the angle of repose for a height of 200 feet, the pressure will be little increased if this extends to 500 feet ; and conversely if a wall of equilibrium is designed to support an indefinite slope, when the actual height of surcharge is 200 feet, the gain in stability will be so slight that some additional area of cross-section will also be required for adequate stability.

Thursday, 19th December.

THOMAS MONRO, President, in the Chair.

The following candidates, having been balloted for, were declared duly elected as:

ASSOCIATE MEMBERS.

ALEX. C. MCCALLUM, JAMES MILNE.

ASSOCIATE.

FREDERICK G. B. ALLAN.

STUDENTS.

ARCHIBALD R. HOLMES, ARTHUR J. MATHESON,
FREDERICK L. JERMYN, E. ST. JOHN MAUNSELL.

The following was transferred from the class of Associate Member to the class of Member:

CECIL BRUNSWICK SMITH.

The following were transferred from the class of Students to the class of Associate Members:

WILLIAM FRANCIS CAMPBELL, ALEX. SCOTT DAWSON,
ERNEST S. MATTICE.

The discussion on Mr. Kerry's paper on "Some Open Questions on the Minor Problems of Railroad Building," and on Mr. Dawson's paper on "A New Method for the Design of Retaining Walls," occupied the evening.

Thursday, 9th January, 1896.

HERBERT WALLIS, Vice-President, in the Chair.

The following having been balloted for, were declared duly elected as :—

HONORARY MEMBER.

W. C. McDONALD.

STUDENT.

WALTER MOFFATT SCOTT.

Transferred from the class of Associate Member to the class of Member :—

ALEX. KING KIRKPATRICK, ARTHUR TRISTRAM PHILLIPS.

Transferred from the class of Student to the class of Associate Member :—

CHARLES BURRARD KINGSTON, JOHN KING MACDONALD.

Paper No. 109.

CEMENT TESTS.

By CECIL B. SMITH, MA.E., M.CAN.SOC.C.E.

PAPER II.

FROST TESTS.

In a previous paper, read before the Society, the writer promised to place before its members the results of certain frost tests, which were being made at that time.

They are now given, in hope that they may be of some interest to those engineers who are contemplating the building of cement mortar masonry, or cement concrete in cold weather.

Method of procedure.—The briquettes were all made in the same manner, the 1 to 1 mixtures having 18 per cent. of water, and the 3 to 1 mixtures 15 per cent., being purposely greater than the amount used in ordinary laboratory tests, so as to get the mortar softer, and resembling more closely the condition in which masons use mortar in ordinary construction, as the effect of frost may be greater on soft mortars than on dry ones.

The briquettes were all rammed into the moulds in 3 layers, and the briquettes to be subjected to frost tests were immediately put outside on a window-sill. In a few hours, after the briquettes were frozen hard, they were removed from the moulds, and left exposed on the window-sills for two, three, or four months, care being taken to keep the snow swept off so as to allow the frost to have its full effect.

The tables given speak for themselves, and probably each engineer will draw special conclusions of his own; the writer will only mention a few points that seem obvious to him.

I. FOUR MONTHS TESTS.

It would appear, from these tests, that it is quite safe to build masonry work in November, in Montreal climate, when the materials are mixed and exposed to the air at about the freezing point. The proportion which the strength of the frost tests bears to the submerged ones is about that which would be obtained under the most favourable circumstances. The briquettes were all firm, smooth, and hard on the surface, and although subjected to four months of severe frost in an exposed position, they did not seem to have been at all damaged.

II. THREE MONTHS TESTS.

These were all made in December, and the coldest days were purposely selected. Yet the only briquettes which were blown in pieces were those made from two very inert, slow-setting, poor Canadian natural cements. The two other natural cements (one Canadian, the other Belgian) were quicker setting, and stood the test well. With the Portland cements, the diminution in strength is more apparent than real, the proportion of 90 to 164, which is the average of 11 brands, is really between briquettes $\frac{3}{4}''$ to $\frac{7}{8}''$ square, and briquettes $1''$ square, the frost specimens being weathered off.

It is reasonable, however, that a briquette $1''$ square exposed on three sides to the direct action of the frost, is rather more severely tested than mortar would be if placed in a wall; even the bottoms of the briquettes resting freely on the stone window-sills were largely uninjured, and the centres of all the briquettes appeared uninjured. As a result of these experiments, the writer would feel perfectly safe in laying cement mortar in December, with Portland or active natural cements, in weather 10° to 15° above zero, and in the most exposed situations, expecting in the spring, to find $\frac{1}{4}''$ to $\frac{1}{2}''$ disintegrated at exposed joints, and needing re-pointing, or, better still, the pointing could be left till spring, and done once for all.

III. TWO MONTHS TESTS.

These tests were much more severe in their nature; the sand and cement were exposed for hours in the open air, in small quantities, until they were absolutely down to the temperature of the outer air, and in the cold water and salt-water series the water was also exposed until it was, in three cases, actually below the freezing point, being in a slushy condition.

These materials were put together in the laboratory as rapidly as possible, and exposed again at once, the usual interval being about 6 minutes, and the actual temperature of the mortar just before exposure having reached about 33° or 34° F., while in the hot water tests the mixture rose, on an average, to 58° or 60°, just before exposure, which was just about laboratory temperature.

The experiments are hardly extensive enough to be fully conclusive, being made only on seven brands of cement, but they point clearly to the advantage of the use of salt. Those briquettes made with salt showed good strength and little injury; although made with materials, at low temperatures exposed in severe cold, they seemed to be chiefly affected only on the surface.

On the other hand, the use of hot water does not seem to be of any advantage, particularly in Portland cements; a reason advanced by one writer for this fact was, that the bringing together of materials in a mortar, at widely divergent temperatures, exerted a prejudicial effect on the cement, hindering proper crystallization, and that the use of materials at as nearly as possible the same temperatures would produce more rapid and stronger action. The effect of hot water on natural cements is not so disappointing, but does not show much increase over the strength of similar specimens made with cold water.

The general result of these experiments, to the writer's mind, points to the idea that in any weather, in winter, not extremely cold, say not lower than + 15° F., masonry work can be laid with cold sand, cold cement and cold water, provided the natural time of set of the cement is not more than five or six hours, and that by the addition of about 2 or 3 per cent. of salt to the water, the same work may be done in weather down as low as zero, which is as cold as men will work. The disintegration will not extend probably deeper than $\frac{1}{4}''$ to $\frac{1}{2}''$—the remainder of the mass being quite sound.

By what process cement sets, after it has, in a few minutes, been frozen solid, and remains frozen for months, the writer will leave to others to explain; but set it certainly does, without ever having been thawed out.

4 MONTH TESTS.

BRIQUETTES MADE DURING MONTH OF NOVEMBER, 1894.

Date of Exposure.	No. of Brand. (See Paper I.)	Ordinary time of setting.		Mixture.	Temperature at time of mixing, in degrees Fah.			Temp. of outside air.	Time elapsed from mixing to time of exposure, in minutes.	Tensile Strength.		*Remarks concerning exposed Specimens.*
		Initial.	Full.		Lab. air	Water.	Materials.			Lab. tests.	Exposed specimens.	
		h. m.	h. m.									
Nov. 14th.	1 old sample	6 00	12 00	1 to 1	59°	56°	59°	36½°	10	321	236	All these were of natural cements. They were brought in and kept 2 or 3 hours before testing, and allowed to warm so as to drive out the frost, and insure a test not being made on a frozen specimen. There were no external signs of any effect produced by 4 months exposure.
Nov. 16th.	2	5 30	—	"	64°	60°	64°	53°	7	302	220	
Nov. 2nd.	2	45	2 45	"	65°	58°	60°	60°	10	484	250	
Nov. 1st.	15	1 00	2 30	"	65°	62°	64°	66°	10	541	237	
			Aver	ages	63°	59°	62°	51°	9¼	412	236	
Nov. 5th.	3	5 00	20 00	3 to 1	68°	65°	65°	37°	12	143	147	All these were of Portland cement. They were, when necessary, brought into the laboratory and kept there for 2 or 3 hours before being tested, so as to insure that no tests were made on frozen briquettes. No signs of the effect of frost were visible on any of the specimens.
Nov. 26th.	4	37	3 10	"	64°	58°	57°	29°	7	236	200	
Nov. 27th.	5	1 00	5 00	"	61°	62°	56°	36°	9	237	183	
Nov. 28th.	*6a*	2 00	6 30	"	68°	64°	63°	28°	8	222	128	
Nov. 6th.	8	3 20	6 30	"	63°	59°	60°	42°	9	172	114	
Nov. 9th.	9	13	2 00	"	64°	57°	61°	32°	11	194	182	
Nov. 13th.	10	25	50	"	62°	57°	57°	35°	7	174	176	
Nov. 19th.	11	30	1 00	"	60°	55°	57°	34°	8	153	141	
Nov. 21st.	12	25	3 00	"	66°	52°	58°	39°	9	119	102	
Nov. 20th	14	20	2 30	"	61°	54°	58°	26°	9	131	125	
Nov. 22nd	19	2 40	7 40	"	58°	54°	56°	37°	7	253	387	
			Avera	ges	62½°	58°	61°	34°	8⅔	185	171	

3 MONTH TESTS.

BRIQUETTES MADE DURING THE MONTH OF DECEMBER, 1894.

Date of Exposure.	No. of Brand. (See Paper I.)	Ordinary time of setting.		Mixture.	Temperature at time of mixing, in degrees Fah.			Temp. of outside air.	Time elapsed from mixing to time of exposure.	Tensile Strength.		*Remarks concerning exposed Specimens.*
		Initial.	Final.		Lab. air.	Water.	Materials.			Lab. Test.	Exposed specimens	
Dec.		h. m.	h. m.						min.			
26	1	6 00	12 00	1 to 1	62°	60°	58°	+ 7°	5	247	0	Briquettes frozen long before set could take place, all blown to pieces by frost.
24	(old)2	5 30	......	"	60°	52°	53	+10½°	7	198	0	ditto
1	2	45	2 45	"	65°	60°	60°	+19½°	8	190	233	Seemingly quite sound, but broke irregularly as to loads and position of fractures.
1	15	1 00	2 30	"	61°	56°	56°	+16°	7½	484	311	Practically sound, some slight cracks on the surface.
Average of No. 2 & No. 15					63°	58°	58°	+18°	7¾	337	272	
Dec.												
3	3	5 00	20 00	3 to 1	60°	60°	56°	+17°	7	108	101	About 1-16" on the surface, disintegrated, the remainder quite sound.
31	4	37	3 10	"	60°	56°	56°	+19°	6	204	85	ditto
31	5	1 00	5 00	"	48°	45°	46°	+14½°	6	218	111	ditto
31	6*a*	2 00	6 30	"	52°	48°	49°	+ 8½°	6½	247	47	Three of these disintegrated for 1-16" on outside, the other two injured to the very centre, average of three being (72).
8	8	3 20	6 30	"	69°	63°	63°	+21°	6½	191	163	Seemed perfectly sound and solid.
10	9	13	2 00	"	68°	64°	64°	+14°	8	151	113	These during a warm spell of three days remained quite soft, not setting at all; when tested, they showed a slight weathering on the top surface.
18	10	25	50	"	57°	53°	53°	+18½°	7½	132	154	Seemed perfectly sound and solid.
27	11	30	1 00	"	70°	65°	65°	+ 7°	7	107	59	Disintegrated for ¼" on top and sides, remainder solid looking.
28	12	25	3 00	"	61°	55°	59°	+ 9°	6	89	23	Disintegrated for ½" on top and sides, remainder solid looking.
31	14	20	2 30	"	50°	50°	54°	− 7°	7	131	49	ditto
29	19	2 40	7 40	"	65°	61°	61°	− 6°	6	223	87	Only 1 briquette was disintegrated on the surface, but all were weak and brittle, crumbling if rubbed with the fingers.
			Averages		60°	53°	54°	+10½°	6⅔	164	90	

2 MONTH TESTS.

(With cold water.)

BRIQUETTES MADE DURING THE MONTH OF JANUARY, 1895.

Date of Exposure.	No. of Brand, (See Paper I.)	Ordinary time of setting.		Mixture.	Temperature at time of mixing, in degrees Fah.			Tem. of mixt're just before exposure.	Temp. of outside air.	Time elapsed from mixing to time of exposure.	Tensile Strength.		Remarks concerning exposed Specimens.
		Initial.	Full.		Lab. air.	Water.	Materials.				Lab. tests.	Exposed specimens	
		h. m.	h. m.							min.			
Jan. 14	2	45	2 45	1 to 1	61°	32°	+19°	40°	+18°	6	295	21	Practically all blown to pieces, the solid core of two briquettes giving 105 lbs. = 21 lbs. average.
5	15	1 00	2 30	"	57°	36°	+26°	38°	- 3°	6	330	87	All the exterior blown to pieces, int'r solid
			Averages		59°	34°	+22½°	39°	+ 7½°	6	312	54	
Jan. 21	3	5 00	20 00	3 to 1	63°	32°	+14°	34°	+13°	6½	86	0	All soft and crumbling. No strength at all.
24	8	3 20	6 30	"	57°	32°	+ 5°	36°	+ 5°	9	214	5	Cem. frozen when mixed 6′ mixed by hand, a very severe test; briquettes appeared firm on surface, but crumbled when touched.
29	9	13	2 00	"	60°	32°	+20°	37°	+18°	6½	133	92	Disintegrated on top for 1-16″; remainder solid.
Feb. 5	10	25	50	"	55°	34°	-11°	30°	-11°	6	145	39	This mortar frozen when mixed, mixed by hand on table, a very severe test, briquettes appeared firm on surface, but weakened all through.
			Averages		59°	32½°	+ 7°	34°	+ 6°	7	144	34	
	Average of		Nos. 3,	8 and 9	60°	32°	+13°	36°	+12°	7	144	32	

2 MONTH TESTS.

(With hot water.)

BRIQUETTES MADE DURING THE MONTH OF JANUARY, 1895.

Date of Exposure.	No. of Brand. (See Paper I.)	Ordinary time of setting. Initial.	Final.	Mixture.	Temperature at time of mixing, in degrees Fah. Lab. air.	Water.	Materials.	Tem. of mixt're just before exposure.	Temp. of outside air.	Time elapsed from mixing to time of exposure.	Tensile Strength. Lab. tests.	Exposed specimens	*Remarks on exposed Specimens.*
		h. m.	h. m.							min.			
Jan. 18	2	45	2 45	1 to 1	64°	125°	35°	68°	+11°	6	428	109	Badly blown on exterior for ⅛", but interior still solid.
5	15	1 00	2 30	"	57°	126°	30°	65°	+ 3°	6	250	23	Top surface blown off for ¼", interior solid looking.
			Aver	ages	60½°	125½°	32½°	66½°	÷ 7°	6	339	66	
Jan. 21	3	5 00	20 00	3 to 1	63°	125°	18°	61°	+15°	6	85	0	All soft and crumbling, no consistency at all,
23	8	3 20	6 30	"	64°	110°	20°	59°	+20°	6½	99	47	Set very slowly in laboratory, those exposed were neither frozen nor set after 4 hours.
30	9	13	2 00	"	63°	119°	18°	59°	+18°	5½	109	88	Disintegrated for about ⅛" on top, remainder solid.
Feb 5	10	25	50	"	55°	115°	-11°	54°	-11°	7	132	21	Slightly disintegrated on top, and weakened all through.
			Aver	ages	61°	117°	+11°	58°	+10½°	6	106	39	
	Avera	ge of	Nos. 3,	8 and 9	64	118°	+19°	60°	+18°	6	98	45	

2 MONTH TESTS.

(With 2 per cent. of salt in the water.)

BRIQUETTES MADE DURING THE MONTH OF JANUARY, 1895.

Date of Exposure	No. of Brand. (See Paper 1.)	Ordinary time of setting.		Mixture.	Temperature at time of mixing.			Tem. of mixt're just before exposure.	Temp. of outside air.	Time elapsed from mixing to time of exposure.	Tensile Strength.		*Remarks on exposed Specimens.*
		Initial.	Full.		Lab. air.	Water.	Materials.				Lab. test.	Exposed specimens	
Jan.		h. m.	h. m.							min.			
18	2	45	2 45	1 to 1	64°	32°	22°	41°	11°	6	320	73	Blown on surface for about ⅛", interior solid.
9	15	1 00	2 30	"	58°	40°	9°	42°	9°	6	280	143	Slightly blown on bottom, other fine crack on top, otherwise solid.
			Aver	ages	61°	36°	15½°	41½°	+10°	6	300	108	
Jan.													
21	3	5 00	20 00	3 to 1	65°	29°	25°	39°	25°	6	101	39	Exterior worn with loose sand, but interior hard and firm, water was slushy at time of mixing.
28	8	3 20	6 30	"	56°	30°	13°	30°	12°	6½	183	224	In perfect condition, water was slushy at time of mixing.
31	9	13	2 00	"	57°	30°	17°	30°	19°	6	105	92	One briquette badly affected, and others quite sound.
	Aver	age of	(3)		59°	30°	18°	33°	17°	6	130	118	No. 10 is not tested.

NOTE.—Each test recorded in this table is the average of 5 briquettes, all briquettes rammed moderately, in 3 layers, with an iron hammer having ½" square end, and weighing about ½ lb.

DISCUSSION.

Mr. Marceau had found it necessary in 1893 to rebuild the spray or wing walls at the upper end of the Grand Lock on the Grenville Canal. The work had to be done during winter, so as not to interfere with navigation. The old walls that had to be taken down were dry masonry walls, resting on a cribwork foundation, each being about 90 feet long, 20 feet high, and 7 feet wide at base. The weight of the stone had more or less crushed the timber work, and the action of the frost on the filling behind had pushed the whole out of line, the walls being in danger of being overturned. To prevent the recurrence of a similar accident, the timber foundation was done away with and cement masonry walls built up from the bottom of the canal, their height being about 29 feet, and their width at base 10 feet 6 inches. The laying of the masonry was in March, and some of the work was performed with the thermometer as low as 14° below zero. All the materials except the cement were heated. A box $10' \times 5' \times 3'$ was first half filled with sand, a rubber hose coiled on top of this so as to cover most of the layer, and sand added up to the top of the box, which was carefully covered. The tube was then connected with a boiler, and steam allowed to pass through it for an hour or so. The free end of the tube was dropped into a barrel filled with water, which was thus also heated by the escaping steam. When a stone had to be laid it was lifted up by means of a derrick and a jet of steam turned on it. With the pressure in the boiler at 70 lbs., the stone was not only heated to a certain depth, but its faces were at the same time made perfectly clean. The top of the course already laid was also heated by a steam jet just before laying the stone in hand. While this was being done the mortar was being prepared, care being taken not to mix more at a time than was required for each operation. The speaker was under the impression that most of the mortar thus made with hot sand and water, and used in the manner described, did not freeze before setting. Some, however, which being made after the materials had cooled to certain extent, and laid towards the evening, must have frozen before having time to set, but this did not seem to have had any injurious effects. The face joints in the portion of the wall Mr. E. Marceau.

which was built in very cold weather were later found to be disintegrated to a depth of an inch or more. These were jointed with pure cement as soon as the temperature permitted. The cement used was Brooks' and Shurbridge's Portland, which sets in from two to three hours in ordinary weather. The speaker examined these walls lately and found no trace of settlement in them, which shows that the setting of the mortar had been perfect.

Prof. C. B. Smith. Mr. Smith remarked that the mortar would always freeze before it was set.

The President. The President asked Mr. Marceau if he had any reason to suppose that it had frozen before it set.

Mr. E. Marceau. Mr. Marceau replied that the seat of the block to be laid as well as the block itself having been well heated, it was most probable that the mortar did not freeze except in the case already stated, and it seemed quite possible to prevent the mortar freezing if proper precautions were taken.

The President. The President asked what size the building stones were.

Mr. E. Marceau. Mr. Marceau replied about two or three feet. Sometimes as much as five feet.

Prof. C. B. Smith. Mr. Smith stated that in the tests which were made at the University it was found that cold materials mixed together gave better results than when only one of the materials was heated. The addition of salt was an improvement, but with all cold materials it was absolutely certain that cement would freeze in five minutes; and as it perhaps would not thaw for four or five months, there would be a chemical action while the water was in the shape of ice.

The President. The President remarked that Mr. Smith's tests showed some specimens to be injured on the surface to a depth of about a quarter of an inch, and that the strength of cold water mixtures was greater than when the cement was mixed with heated water.

Prof. C. B. Smith. Mr. Smith explained that the temperature of the air at the time of putting the specimens out, and the temperature of the mixture just before exposure, were given. In the hot water tests the average was about 68° Fah. In other words, the hot water and the cold sand brought the mixture to a normal temperature, but they froze in about five minutes after exposure to the cold.

Prof. C. B. Smith. Mr. Smith asked if in the work mentioned by Mr. Marceau, the steam was applied to the stones long enough to increase their temperature?

Mr. E. Marceau. Mr. Marceau replied that they were kept under the steam jet for five or ten minutes in very cold weather. This, he thought, would raise the temperature of the stone for a depth of one-half to three-quarters of an inch. The operation was done in such a way that the faces of the stone which were to be in contact with the mortar had not time to get cold before the block was in place.

The President remarked that the stone being a poor conductor prevented the frost penetrating the mortar. The President.

Mr. Kennedy stated that some two or three years ago he had had a conversation with the Chief Engineer of the U. S. Government staff at the Sault Ste. Marie Lock, on this question. This gentleman had made a great many experiments, and there was one room in connection with the works in which there was a man engaged in making tests constantly. The conclusion reached from their tests was that there was no deterioration from the effects of freezing. Mr. Kennedy had not paid special attention to this matter, but the gentleman mentioned had expressed his surprise at the result of his experiments. The tests made were similar to Mr. Smith's and under nearly the same conditions. The cement used was Portland cement. Mr. W. Kennedy

Mr. Marceau remarked that it seemed quite certain that in the work he had referred to, some of the mortar had frozen before it had time to set. Mr. E. Marceau.

Mr. Smith said the question was whether the frost ever went in any further than the surface. Prof. C. B. Smith.

Mr. Marceau stated that the joints were sounded before being pointed, and were found quite hard beyond the disintegrated portion. Mr. E. Marceau.

Mr. Smith remarked as to the disintegration of these joints, it was in the middle of March they were laid, and probably the sun's heat was sufficient to thaw them out before they were well set. Prof. C. B. Smith.

Mr. Marceau replied that he was not on the ground himself constantly during March, and the person in charge of the work had not noted this point. However, the sun's heat was undoubtedly great on certain days. Mr. E. Marceau.

Mr. Smith asked what proportion of cement was used in the mortar? Prof. C. B. Smith.

Mr. Marceau replied that the mortar was in the proportion of one volume of cement to two of sand. Mr. E. Marceau.

Mr. Smith pointed out that the difficulty of connecting the results which he had obtained with actual practice was that the proportions used by the cement testers and the manufacturers were not the same. In the laboratory the proportions have been by weight, whereas it has been used by volume and sold by weight. Would it not be possible that the relation of weight to volume be stated for each brand, Prof. C. B. Smith.

so that units of weight might, by multiplying by a constant, be converted into units of volume, or *vice versa?*

Mr. Jos. Rielle

Mr. Rielle stated that a good deal of brick work in Montreal has been laid in winter with good results. The speaker had occasion to oversee for some time the construction of raceways on the Beauharnois Canal, and one of these was built in the month of January. The cement was mixed in a shop and the sand was heated; the sand outside was kept warm and the stones brushed thoroughly clean. The whole work was put through in that way, and proved very satisfactory. In the spring of course all the joints had to be pointed, but so far as the speaker knew there was no difference between the one built in mid-winter and those built in summer.

Mr. H. Irwin.

Mr. Irwin remarked that during the last ten years a large number of masonry works of different sorts had been built on the Canadian Pacific Railway, which had included a large quantity of cement masonry built when the temperature was down to zero, and certainly some of them were built without taking any precautions whatever in the way of warming anything, care only being taken to clear all snow or ice off the stones. The materials must have frozen very rapidly. In one case, where everything was built with cold materials except that the water was not allowed to freeze, it was found that only a fraction of an inch scaled off in the spring. In depositing mortar in extremely cold weather, the cold has, to a certain point, more effect on warm mortar than it has on cold mortar, because in using warm cement in cold air, the effect of the cold is worse on account of the change of volume due to the change of temperature. There certainly have been many works constructed in cold weather with cold mortar, of which not more than half an inch scaled away. With regard to cement deposited that was warmed under good conditions and had the stone set quickly on it, it was possible in setting that the effect of the cold on the outer parts was sufficient to cause the mortar to disintegrate immediately, as it was not subject to pressure on one side, while the pressure on the inside may have kept it from disintegrating. Cement has, in general, set much better under heavy pressure than under light pressure.

Prof. C. B. Smith.

Mr. Smith stated that the two months during which the tests were made were very cold. In the last test in the table the mortar was frozen when mixed. The water was 34° and the sand and cement were 11° Fah. below zero. In attempting to mix them with the mixer the mass froze solidly. It was then taken out, pounded to a powder, and placed in the mould. The test of these showed that they were not so much disintegrated, but that they had not properly set. Between the cold water tests and the cold water with salt, Nos. 3, 8 and 9 in the table, it will be noticed that the average of those in the open was 118, and the average of those put in the tanks was 130. In the one case only, those exposed outside stood more than those in the tank. The speaker had been in the habit of using salt in all actual winter construction.

Mr. Irwin remarked that the experiments which were being made at McGill College as to the temperature of the ground and the rate at which the cold goes down into the ground, have shown that during one fall the frost did not get into the ground to any depth for a long time, but that it went straight down to a depth of three feet during a heavy rain. Mr. H. Irwin.

Mr. Kennedy remarked that in laying pipes for waterworks it has been found that the spring was the worst time for the freezing of pipes. The water carried the frost with it. Mr W. Kennedy

The President asked was it not a fact that sugar was used in cement? Some of the hardest cements of the ancient times he believed were supposed to have been set with it. The President.

Mr. Irwin stated that in the taking down of part of Carrickfergus Castle, which he supposed was a limestone structure, he remembered distinctly some cases in which the stones themselves broke instead of the mortar joints giving way. The mortar was of hydraulic lime. Mr. H. Irwin.

The President remarked that chimneys built with cement were very apt to crack. The President

Mr. Irwin stated that it had been often advocated that arches had much better be built in a good mixture of lime and cement mortar, as the mixture would set more slowly, and allow the arch stones to settle without cracks. Mr. H. Irwin.

Mr. Marceau stated that in going through the town of Laprairie a number of years ago, his attention had been attracted to the actions of some thirty men trying to pull down a piece of an old wall about thirty feet square. The width near the bottom of it had been reduced to three or four feet, and a rope tied to the top. He watched the men tugging at this rope for an hour or so without success. He could not say whether they had succeeded in the end, or whether they had to use explosives. This wall had formed part of an old fort built during the French regime. Mr. E. Marceau.

Mr. Smith remarked that in most of the old lime mortars there was a considerable percentage of clay, which increased their strength and in time made them hydraulic. Prof. C. B. Smith.

Mr. Irwin stated that the custom over twenty years ago in Montreal Mr. H. Irwin.

was to use not the gray limestone but the black limestone, which took much more fuel to burn. The burning of black limestone has cost more than grey. The black limestone gave an hydraulic lime.

Mr.E.Marceau. Mr. Marceau remarked that the practice in some parts of the Province of Quebec was to mix mortar in the fall and bury it in trenches well protected against frost. When used in the spring this mortar had the consistency of putty.

OBITUARY.

Patrick Kennedy Hyndman, whose death occurred on July 10, 1895, at Sarnia, Ontario, was the youngest surviving son of the late Mr. Henry Hyndman, and was born at Lunderston, near Goderich, on April 8, 1842. He was an articled pupil of the late Mr. T. N. Molesworth, C.E., P.L.S., and for three years was second assistant engineer on the railway extension to the harbor of Goderich, and work in connection with the harbor there. Mr. Hyndman passed his final examination as a Provincial Land Surveyor in Toronto in 1862. In 1865 he went to India, and entered the service of the Government, after three years' construction work on the East Indian Railway under Mr. (afterwards Sir) Henry Peveril LeMesurier, M.Inst.C.E. He remained in the service of the Government for over ten years, having charge, as executive engineer 4th, 3rd and 2nd grades respectively, of canal, bridge, road, civil building and general railway construction work. He was compelled to leave India in 1878 on account of ill-health, and retired under special pension and bonus.

Mr. Hyndman afterwards came out to this country, where he had charge of preliminary work on the construction of the Canadian Pacific Railway in the Rockies, and the S. S. V. Railway in the vicinity of Prince Albert. In 1884 he brought his family out from Scotland and settled in Ottawa, where he built up a good private practice, and in 1887 was offered a position on the Government Railway in Cape Breton, but after accepting it, was unable to perform a day's work on account of rheumatism. He afterwards recovered sufficiently to take charge of a preliminary survey for a private line in Cape Breton, but was again compelled to give this up, and had to return home and take to his bed, from which he never again rose.

Mr. Hyndman leaves a widow and four children (a married daughter and three sons). He was a member of the Institution of Civil Engineers and was elected a member of the Canadian Society of Civil Engineers on January 20, 1887.

Edmund Walter Plunkett came to Canada, from Ireland, in about the year 1855, and obtained an engagement on the engineering staff of the Grand Trunk Railway, which he held for probably ten years. He then, in partnership with Mr. James Brady, opened an office in Montreal for general practice as surveyors and engineers. A large map of Montreal by "Plunkett and Brady," exhibited the most complete survey of the city that had been made up to that time. The firm of Plunkett and Brady was dissolved somewhere about 1868. In the following year Mr. Plunkett was associated with his brother-in-law, the late Mr R. E. Cooke, C.E., in planning improvements for the water supply of Montreal. He subsequently became contractor for the Hamilton and Lake Erie Railway, on which and other undertakings in Western Ontario he was engaged till 1875, when he joined Mr. Walter Shanly in the construction of the Western Counties (Nova Scotia) Railway. Mr. Shanly resigned his interest in 1877, and Mr. Plunkett continued to be connected with it, to the exclusion of almost all other engineering engagements, for the greater part of the remainder of his life. In 1880 he undertook for Mr. W. Shanly a survey for a tunnel under the St. Lawrence between Hochelaga and Longueuil. Mr. Plunkett spent many of the later years of his life in Europe, chiefly in London. He went to Vancouver, B.C., in December, 1891, and died there in September, 1893, aged about 61 years. He was elected a member of the Canadian Society of Civil Engineers January 20, 1887.

CHARLES SPROATT was born in Toronto on the 21st of June, 1835, and educated at Upper Canada College. After leaving college he entered on the study of his profession with Mr. John A. Tully, Civil Engineer and Provincial Land Surveyor. Upon the completion of his studies he accepted a position on the G. T. Ry. construction work, west of Toronto, with Mr. Frank Shanly, C.E. In 1869 he was appointed Asst. Engineer on the location of the Toronto, Grey & Bruce Ry., and afterwards appointed Division Engineer on construction, and in 1871 he was appointed Asst. Engineer of the entire road under Mr. Wragge. He occupied this position until 1877, when he was appointed Asst. Engineer under Mr. Frank Shanly, City Engineer of Toronto, to make the necessary surveys for the proposed Trunk sewer. In 1878 he was appointed Chief Engineer of the Georgian Bay and Wellington Ry, upon the completion of which he went to the North-West and had charge of a party on the C.P.R. Division through the Rocky Mountains. He was then employed on the right of way for the same Company through the prairies. He afterwards went into partnership in the North-West, the firm being Gossage, Sproatt and Thompson. In 1883 he was appointed City Engineer of Toronto, which position he held until 1889, when, owing to failing health, he was given leave of absence and went for a trip to Europe. On his return, although his health was still far from good, he accepted a position as Deputy City Engineer, but, owing to continued bad health, he resigned his position and moved with his family to the North-West Territories, and died at Innisfail, N.W.T., on December 27, 1895. His failing health was no doubt directly attributable to the arduous and continuous work and worry during the six years he was City Engineer of Toronto. It was during this period that the real estate "boom" was at its height, and the work thrown upon Mr. Sproatt's shoulders was very onerous and responsible. During his term as City Engineer he constructed some very important works, amongst others the King Street subway and the Don River improvements. Mr. Sproatt was universally liked by those with whom he was associated professionally. His amiable disposition and willingness to assist others endeared him to all. He was elected a member of the Can. Soc. C.E. on January 20, 1887.

Arthur Mellen Wellington * was born in Waltham, Mass., December 20, 1847. He was descended on his father's side from an old New England family, which had resided on a rocky hillside farm in the town of Lexington, Mass., since Colonial times. He graduated at the Boston Latin School, and then, when only sixteen years old, began the study of civil engineering in the old-fashioned way, by apprenticeship to a practicing engineer. For the three years from 1863 to 1866, he was an articled student in the office of John B. Henck, of Boston, well known to engineers as the author of "Henck's Field-Book." His first work in civil engineering after leaving Mr. Henck's office was in the engineering corps of the Brooklyn Park Department, under Mr. Frederick Law Olmstead, where he served as leveler and assistant engineer. In 1868 he obtained his first position in railway work, a field in which he was to win enduring fame. This was on the Blue Ridge Railroad in South Carolina, where he remained for a year as transit man, having charge of a locating party. He then went to the Dutchess and Columbia Railroad in New York, and for nearly a year served on that road as an assistant engineer. In 1870, when twenty-three years of age, he was placed in charge of a division of the Buffalo, New York and Philadelphia Railroad, and, notwithstanding his youth, he was soon

* This notice is taken with some modification and abridgment from the *Engineering News* of May 23, 1895.

advanced to the position of Principal Assistant. After remaining with that company two and one-half years, he became locating engineer of the Michigan Midland Railroad, and later was Engineer-in-Charge of the Toledo, Canada Southern and Detroit Railroad.

The panic years of 1873-4 put a sudden stop to railway construction, and Mr. Wellington, in common with hundreds of other engineers, found his occupation gone and no demand for his services in any new position. In his application for membership in the American Society of Civil Engineers, made in 1881, he said: "1874-'78, was engaged in miscellaneous professional business and literary occupations, more interesting than lucrative, and not always particularly interesting." In later years, however, he was accustomed to refer to this period of enforced idleness—so far as idleness was possible to a man of his restless energy—as a blessing in disguise. Hard as it was for the young engineer to leave the professional work in which he was so intensely interested, and making such satisfactory advancement, it caused him to use his enforced leisure for the study of the broader problems in connection with his profession, and to lay the foundations for the more important work of the later years of his life.

Mr. Wellington's first literary venture was made in 1874, when he was only twenty-seven years of age. It was "The Computation of Earthwork from Diagrams," a book which was the outcome of the methods he had worked out for expediting his computation on the railways on which he was engaged. This book was very favourably received, and in the intervals during the years 1874-78, when other occupations failed him, it was natural that he should again turn his attention in the direction of contributing to the literature of his profession, and upon the subject in which he had had the most experience—railway location. His great work, and that by which his fame as an engineer was established, "The Economic Theory of the Location of Railways," was begun in 1875, as a few notes in preparation for an anticipated location. It was afterward expanded into a magazine article and was first published in the "Railroad Gazette" in the latter part of 1876, as a series of articles on "The Justifiable Expenditure for Improving the Alignment of Railways." These articles were reprinted in book form in 1877, and the attention of engineers and railway men was at once attracted to their writer as an engineer of uncommon brilliancy and ability.

In 1878 Mr. Wellington accepted the position of Principal Assistant to Mr. Chas. Latimer, Chief Engineer of the New York, Pennsylvania and Ohio Railway. His duties here were, from one point of view, less to his taste than the work of railway location. Nevertheless, the three years spent on this work gave him an opportunity to gain experience in railway operating details and to acquire a vast fund of information, of which at a latter date he made good use.

In the summer of 1878, through the courtesy of Mr. Charles Paine, then Chief Engineer and General Manager of the Lake Shore and Michigan Southern Railway, Mr. Wellington carried out an extended series of experiments on the resistance of rolling stock, the results of which were presented in a paper read before the American Society of Civil Engineers, on the 15th January, 1879. Those experiments were made chiefly by dropping cars down a known grade, and had much influence in establishing formulas for train resistance at low velocities. In the following winter he carried out a series of tests on journal friction at low velocities, the results of which, however, were not made public until 1884, when they were embodied in a paper read by him before the American Society of Civil Engineers. It is an excellent illustration of the thoroughness and absorbing interest with which he undertook the solution of any engineering problem, that having made the train resistance tests above noted, and finding in the results some elements of uncertainty, he should plan and carry out in the little leisure which his regular duties gave him a further elaborate series of tests to settle the doubtful points.

After spending three years on the New York, Pennsylvania and Ohio Railway, Mr. Wellington accepted in March, 1881, a position as Engineer-in-Charge of Location and Surveys on the Mexican National Railway. Some of the most interesting portions of his work on that line were described in a paper read by him before the American Society of Civil Engineers in July, 1886. During the three years, 1881-'84, Mr. Wellington remained in Mexico, first in the service of the Mexican National Railway, and later as Assistant General Manager and Chief Engineer in charge of location of the Mexican Central Railway, under Mr. Rudolph Fink.

But the work of railway location, congenial as it was to him, he was soon to exchange for an occupation still better suited to his tastes and abilities. In 1884, he returned to the United States and entered the field of technical journalism, becoming one of the editors of the *Railroad Gazette*. His experience in writing books, and as a contributor to various journals, had already familiarized him with literary work, and had revealed an exceptional talent for it, and he entered upon his new field of labor with a zeal and ability which at once attracted attention.

While upon the staff of the *Gazette* he edited the revised edition of the "Car Builders' Dictionary," and his leisure was devoted to preparing for the press the second edition of his work on "Railway Location," which was finally published in the spring of 1887. The value of this work had been well proved by the demand for it. The first edition was soon exhausted, and before the second was issued the price for second-hand copies rose higher and higher until as much as $20 was paid for a single copy.

In January, 1887, Mr. Wellington became one of the editors-in-chief and part owner of the *Engineering News*. The influence of his energy and ability was at once seen in every department of this journal. In his editorial work he combined in wonderful measure the two valuable qualities of originality and industry for which he was conspicuous. If he edited a letter for publication in the correspondence column, it was sure to suggest some idea to him which he would add as editorial comment. If he prepared a note for the *Engineering News* page, it was seldom a colorless recital. Some piquant criticism would be thrown in. His industry was measureless; he never dropped a proposed scheme merely on account of the amount of labor involved, but seemed to regard it rather as a sort of challenge and undertook it with the greater relish.

Mr. Wellington found time during the years following 1887 for occasional service as a Consulting Engineer. Among the more important works on which he gave advice were the elimination of grade crossings at Buffalo, the improvement of railway terminals at Toronto and the foundations of the Board of Trade Building in that city. In the summer of 1888 he made an extended examination of the Canadian Pacific Railway system at the request of President Van Horne, and later gave expert testimony in the famous suit between the company and the Canadian Government, in which the character of the construction taken over by the company was in question. He was a member of the Board of Engineers which examined and approved the estimates of the Nicaragua Canal Company, in 1890. In 1893 he was called before the Massachusetts legislature with reference to the proposed invasion of Boston Common by the West End Street Railway, and at his suggestion the Tremont Street subway, now under construction by the city, was decided upon as the best plan for effecting the desired improvement. The last work which he undertook as a Consulting Engineer was the improvement of the railway lines in the Island of Jamaica, where he spent two months in the spring of 1893. He became a member of the

American Society of Civil Engineers in 1881, and was always among its most enthusiastic supporters. At later dates he was elected to membership in the Institution of Civil Engineers, the American Society of Mechanical Engineers, and the Engineers' Club of New York City.

In the summer of 1892 he took a vacation of three weeks, but instead of leaving the city, as was his usual custom, he devoted his leisure to working out some ideas in thermodynamics which had occurred to him years before. It was characteristic indeed of the man and of his innate love for work that he chose to spend his leisure in such a manner, rather than in pleasure-seeking of the ordinary sort. The result of his study was the invention of an entirely new type of thermodynamic engine, designed to convert heat into mechanical work with a much smaller percentage of loss than the best existing steam engines. Henceforward the development of his invention became the all-absorbing work of his life, and in his earnestness and zeal all thought of care for his health was forgotten. It had always been his habit to work far into the night when the hours of the day were not sufficient to make satisfactory progress with whatever he had in hand, but in his labors upon this latest child of his brain, his eagerness was such that he was no longer able to turn his thoughts away from it, even in the few hours which he allowed himself for rest. Even his iron constitution could not bear up under such a strain, and early in 1894 he found himself physically unable to go on with his work. Entire rest brought temporary relief, but not, unfortunately, the restoration to a healthy condition of the overtaxed organism. During the eighteen months from the first conception of his invention until the failure of his health, Mr. Wellington's contributions to the columns of *Engineering News* became less frequent, until they ceased entirely in May, 1894. He had at that time completed his invention, and had made good progress in experiments as to its practical and commercial development. How great a trial it was to drop work upon it, when so near completion, only those closest to him could realize ; but the sanguine and resourceful temperament which had been his stay in every disappointment was evident here, and he made preparations for the European trip which his physicians advised with the same good humor as if it were a mere pleasure journey, and in entire accord with his inclinations.

While traveling in Norway, in August, 1894, his disease suddenly assumed an acute form, and serious hemorrhage of the kidneys occurred, so persistent as to threaten an immediate fatal termination. It was at length arrested, however, and in September he was sufficiently improved in health to return home. His malady was a rare and peculiar one, baffling the physician's skill. A period of several weeks in which he would apparently make steady progress towards restored health would be followed by a sudden return of hemorrhage and a loss of more ground than had been gained. Such alternations are even more calculated to depress the spirit than a steady downward progress; but all through these trying months Mr. Wellington's sanguine cheerfulness was never failing. On the 15th of May, 1895, an operation for the removal of the diseased kidney was performed with success. But, besides the disease at this point, there was a chronic weakness of the heart, and at 9.30 p. m. on the following day that organ refused to perform its work.

Mr. Wellington was by nature a man of intense convictions, his standard of right was high, and compromise with anything which did not reach that standard was always difficult for him to make. In every cause that appealed to his interest, his inclination was always to espouse whichever side he believed to be right and labor ardently for its success. With such a temperament, it was natural that he should occasionally make enemies, especially among those who knew little or nothing of him personally, and could not therefore understand that no real malice lay behind the quick, cutting remark, the caustic comment or the keen satire from his pen. But to those whose privilege it was to know him intimately, the good-heartedness of the man was always evident.

Mr. Wellington was elected a member of the Canadian Society of Civil Engineers on October 8th, 1891.

LIST OF MEMBERS.

ADDITIONS.

MEMBERS.

Date of Membership.

Fleming, Sandford, C.M.G....Ottawa....................18th June, 1896.
MacDonnell, John Alex......Dept. Pub. Works, Winnipeg..18th June, 1896.

ASSOCIATE MEMBERS.

Laird, Robert................Restigouche & Vict. Ry.,
Campbellton, N.B..........18th June, 1896.
Orrock, John Wilson........Beauharnois, P.Q.............18th June, 1896.

ASSOCIATE.

Morrison, Thomas A..........118 St. Peter St., Montreal...18th June, 1896.

STUDENTS.

Macdonald, Wm. Angus.......16 Compton Ave., Halifax,
N.S.....................18th June, 1896.
Renwick, Haliburton Paul....90 Cadboro Bay Rd., Victoria,
B.C.....................18th June, 1896.
Wilkin, F. A................Rossland, B.C..............18th June, 1896.

TRANSFERRED FROM THE CLASS OF ASSOCIATE MEMBER TO THE CLASS OF MEMBER.

Campbell, Arch. William.....St. Thomas, Ont............18th June, 1896.
Symmes, Charles Thomas W. Casilla 1786, Santiago, Chili, 18th June, 1896.

TRANSFERRED FROM THE CLASS OF STUDENT TO THE CLASS OF ASSOCIATE MEMBER.

Amos, Louis Auguste........61 Temple Building, Montreal...18th June, 1896.
Duff, John Andrew..........School of Prac. Science,
Toronto.................18th June, 1896.

CHANGES AND CORRECTIONS.

MEMBERS.

CARRY, H. E. C.................79 Bay St., Toronto.
BERRYMAN, EDGAR..............14 Richmond Square, Montreal.
GOAD, CHARLES EDWARD.........15 Wellington St. W., Toronto.
LEONARD, R. W................Beauharnois, P.Q.
ODELL, C. M..................North Sydney, N.S.
O'DWYER, J. S................Restigouche & Vict. Ry., Campbellton, N.B.
RHODES, A....................P.O. Box 212, Quebec.
SMITH, H. B..................Rossland, B.C.
WALLIS, HERBERT..............239 Drummond St., Montreal.

ASSOCIATE MEMBERS.

CRUMPTON, ARTHUR.............G. T. R., Montreal.
DAWSON, A. S.................Metropolitan Water Bd., 3 Mt. Vernon St., Boston, Mass.
ELLIS, H. D..................Care Messrs. Mackenzie & Mann, Lake Man. Ry. & Canal Co., Gladstone, Man.
FRANCIS, W. J................Care Central Bridge & Engine Co., Ltd., Peterboro, Ont.
HANNAFORD, R. M..............127 Bishop St., Montreal.
McCULLOCH, A. L..............Sarnia, Ont.
MATTICE, E. S................264 Wood Ave., Westmount.
TRUE, ABBOTT.................203 Bruce St., Hamilton, Ont.

STUDENTS.

BAKER, H. C..................Perkin's Mills, P.Q.
COSTIGAN, J. S...............196 St. James St., Montreal.
DYER, W. E. L................258 Wood Ave., Westmount.
GIBBONS, JAS.................Renfrew, Ont.
GREENBERG, LOUIS.............268 Bleury St., Montreal.
HART, O. C...................Cowansville, P.Q.
LAMBERT, FRANK...............The Rectory, Greenwich, Eng.
LONGWORTH, C. H. B...........Charlottetown, P.E.I.
LORDLY, H. R.................35 Dock St., St. John, N.B.
MOONEY, G. W.................Ausable, N.Y.
OGLIVIE, W. M................Cummings Bridge, Ont.
REINHARDT, CARL..............133 Crescent St., Montreal.
RUTHERFORD, F................61 Rosemount Ave., Westmount.
SCOTT, W. M..................Box 352, Charlottetown, P.E.I.
SMITH, G. S..................Ball Engine Co., Newark, N.J.
WIGGINS, T. H................Cornwall, Ont.
WINGHAM, T. H................Bell Tel. Works, Aqueduct St., Montreal.

INDEX.