Correspondence.

Mr. Brodie. Mr. J. B. Brodie observed that the limits fixed by Act of Parliament regarding spans, widths, and headways seemed to have left little choice to the Author in the design of the new Caledonian Railway bridge. Its utilitarian features were well marked; but no great pretensions could be put forward in regard to the æsthetics of the design. With regard to the heavy live and dead loads upon the bridge necessitating large flange-areas for the main girders, on account, as the Author said, of the indifferent proportions in respect of ratio of depth to span there was not much to criticize, the structure being a deck bridge having a series of parallel girders placed at short intervals apart, diagonally braced and stiffened by the floor. Having regard to the statement in the Paper that massive piers were required, and that the gross weights transmitted to the foundations were very great, it would be interesting to know why the Author had preferred a series of cylindrical piers, resting on monolithic foundations, to a continuous solid pier, which would have done away with the cap-sill girders, and would have distributed the load just as well. The solid pier seemed to be less costly, and the argument advanced in support of monolithic foundations, instead of individual piers as originally contemplated, seemed to apply to the continuous pier as against individual cylindrical piers. In the Author's treatment of the question of loads on subaqueous foundations, the sustaining effect of skin-friction had been entirely neglected, and the diminution of the weight of the submerged parts of the pier had been discounted. Although that might have resulted in a greater unit load being permissible on the foundations, it erred on the score of extravagance. It might be pointed out that the foundations of many large railway-bridges had been designed subject to the consideration of the substantial support afforded by those elements, and were doing their work well. The question was doubtless a matter of opinion, and was largely influenced by the financial resources of the company; but as the foundations were among the most important parts of such an undertaking as a large bridge, it was essential that no doubt should exist as to their stability.

Both the new Clyde bridge and the Queen Alexandra bridge were characterized by solidity of construction wherein the best materials had been used and applied in the best manner. Perhaps the most noteworthy feature in the building of both bridges was presented in the erection of the river span of the Queen Alexandra bridge; and the ingenuity displayed in the design of the arrangements for erection, and the bold and skilful tactics of the contractors, were worthy of all praise. Mr. Brodie.

Mr. F. BROOMFIELD, while appreciating the unusual difficulties of the erection of the river span of the Queen Alexandra bridge, and the ingenuity and skill shown in overcoming them, was of opinion that a very unsuitable type of girder had been chosen for the bridge. The question of erection, which should have been the governing factor both in determining the most suitable type of girder to use and in the design of the bridge generally, did not appear to have received serious consideration until after the design had been completed. Not only had that resulted in excessive weight of the bridge proper, but, as the only practicable method of erecting the bridge proved to be by transforming it virtually into a cantilever, a large amount of extra steelwork was required for temporary towers and ties, which might have been economically employed as a permanent part of a much lighter cantilever structure. A great saving would have been effected thereby in the weight and cost of the bridge, and the tedious computations of the stresses and deflections for all stages of the partially erected structure might have been largely avoided; while the time required for the completion of the work might have been considerably reduced. The situation appeared to have been eminently suitable for a cantilever type of bridge, and Mr. Broomfield would like to ask the Authors if there was any reason why that type had not been adopted. Mr. Broomfield

Mr. W. DAWSON agreed with Mr. Matheson's observations that the testing of steel at the steelmakers' works left something to be desired, and that it would be better if the specimens were selected when the material had been delivered at the bridge-building yard. That had been the practice on the London and North Western Railway for the last 25 years or more, and very little inconvenience had arisen in adopting the principle. On the other hand, a large amount of time had been saved by inspectors selecting specimens for testing at one place, instead of having to visit a number of mills where the various sections were rolled. There was also the important consideration that the engineer had thereby greater assurance that proper material was used in the work. With regard to the ties and struts of the web-bracing not being riveted Mr. Dawson.

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Mr. Dawson. together where they crossed, Mr. Dawson thought that would in time give trouble, owing to corrosion. Some years ago his attention was called to the bottom flanges of some roof-girders over Oxford station, where the rivets were spaced 15 inches apart, and in consequence there were insufficient means of keeping the flange-plates in close contact with the angle-bars. The result was that corrosion set in, ultimately forming a long cake of rust $\frac{5}{8}$ inch thick, forcing the plates apart and shearing off the connecting rivets. He thought it was now generally agreed amongst engineers responsible for the maintenance of iron and steel structures that in order to maintain them with efficiency and economy it was essential that they should be kept free from corrosion, otherwise great expense in renewals or reconstruction became inevitable in the course of time, accompanied, as it so often was, by dislocation of traffic and other inconveniences. If it were admitted that corrosion should be prevented, did it not follow that mild steel was more economical to use than wrought iron, seeing that it was practically the same price per ton, and that the allowable working-stress, broadly speaking, was $6\frac{1}{2}$ tons per square inch for tension and compression in the case of the former, and 5 and 4 tons respectively in the case of the latter? In other words, the use of mild steel effected a saving of more than 25 per cent. in first cost.

Mr. Fergusson. Mr. J. C. FERGUSSON, whilst regarding both the Papers as valuable additions to the Proceedings of The Institution, did not agree with the Author of the Paper on the new Clyde bridge on the subject of the foundations, and particularly in his method of determining the frictional resistance of the caissons. Although Mr. Fergusson's views were directly opposed to those of the eminent engineers of the Caledonian Railway, he desired to submit them with all deference. The engineers had ignored "in calculating the stability of the foundations, the possible support to be derived from surface friction": hence the areas of the foundations were determined on the basis of 6 tons per square foot, on river gravel 43 to 48 feet deep. If the skin-friction of the caissons were a negligible quantity, why was it necessary to sink caisson C 1, for example, 48 feet deep? As the piers were calculated as columns, and were tapered up with a batter of 1 in 100, there could be no object in increasing their length; and was not the coarse sand and gravel lying between 30 and 40 feet below the river as good a foundation as the same material at 48 feet below? As the scour of the river would never be 33 feet, might there not have been a saving in dead weight and cost of say 15 feet of the caisson? The fact that the bridge was required to carry nine lines of railway did not necessitate deeper

Mr. Fergusson.

or larger foundations, in proportion, than would have sufficed for one carrying only a double line, over the same span, for the same live load, provided that the foundations were the width of the bridge. With regard to the Author's methods of determining the frictional resistance of the caissons, he would like to make his argument clear by quotations from the Paper (p. 37). "The excavated material was entirely removed from the working-chamber as low as the cutting edge, and the edge was undercut as much as was possible. The men were then withdrawn from the working-chamber, and the air-pressure was reduced, when the caisson ultimately began to sink." Then it was stated that the gross surface friction was equal to the gross weight of the caisson and its loading, minus the gross pressure of the air on the roof of the working-chamber. In Mr. Fergusson's opinion that was not a correct method of determining the frictional resistance of the caissons: it was certainly much more, for this reason:—When these preparations had been made and the compressed air was let out of the working-chamber, the whole of the working-chamber was converted into a suction-chamber, and it then became a matter of sinking the caisson by a vacuum process. The hydrostatic pressure from without the caisson drove before it what little air was left in the working-chamber, leaving a vacuum; and the caisson resting in gravel and water, aided by its own weight, was drawn down. This process for the moment annulled the skin-friction by drawing water down the outer surface of the caisson to the vacuum created in the working-chamber, and displaced water and gravel equal in area to the size of the working-chamber. That there was an enormous suction-effect exerted by this method of operating the caisson was proved by the tremendous displacement and inrush of material from the bottom and sides of the caisson, which filled the working-chamber to the ceiling, 8 feet high. That was a forcible way of breaking up the gravel foundations, but it was not a correct method of testing the measure of frictional resistance of caissons. The main difficulty in founding the caissons at such a great depth was not in the excavation, but in the sinking of them. The engineers foresaw this, and gave them a batter of 1 in 100: they decided to ignore surface support; still they dreaded skin-friction, and provided against it in order to ease the penetration. The duty of a caisson was to carry weight—not to penetrate. The goods-trains in Canada and the United States were often heavier than those in England, yet in places they were carried along foreshores on lines supported by rows of four piles driven 16 feet to 20 feet apart. Where those piles were "driven to a stop" with a 3,000-lb. ram they

Mr. Fergusson. often derived more support from side friction than from their base. That a pointed pile could not be driven in hard sand and gravel deeper than one not pointed afforded further proof that surface friction, through a certain depth, overcame penetration and resisted sinking. From California along the heavily timbered shores of Oregon, Washington, British Columbia and Alaska—a region in which pile-driving was practised largely—one never saw an iron-shod pile, although some of the piles used in the salmon-traps at the mouths of the American rivers were over 100 feet long. Those piles were sawn lengths of trees cut off square at the top and bottom, and driven with the thick end up, which gave a downward taper of about 1 inch in 9 feet. Such piles drove straighter than pointed piles and would carry more weight than a square sawn pile.

Mr. Husband. Mr. JOSEPH HUSBAND remarked that the general effect of the approaches of the Queen Alexandra bridge suffered somewhat by comparison with the girder spans. He would like to ask if, as an alternative scheme, consideration had been given to the question of carrying the roadway of the approach-viaducts on columns instead of arches, and if so, how that had compared economically with the work as constructed? A system of continuous columns from foundation to railway, with the roadway carried on cross girders between the columns at suitable heights, would have lent itself to adequate bracing, and would have presented a more homogeneous and satisfactory appearance than the upper tier of columns resting on the arches. It would also have obviated the necessity for planting rather heavily loaded columns on the crowns of the arches, thereby imposing local concentrated loads which, under severe impact and vibration, might cause trouble with the arches in the future. He presumed, from the curvature of the approaches, that high speeds over the bridge were not likely to prevail: was any limit of speed prescribed? The Paper did not indicate the manner in which the columns supporting the railway of the northern approach were dealt with in order to avoid the traffic on the road from Camden Street. The 4-foot lattice girders between the columns were presumably intended as bracing with a view to stiffen the structure longitudinally. Would not bracing-bars of the full depth, or nearly so, of the pillars have been more efficient in this respect? Difficulty had been experienced in satisfactorily filling the pockets between the brackets of the shoe of the caisson. The caisson appeared to be of somewhat unusual design. The horizontal deck-plates carried over the pockets no doubt added to the lateral rigidity of the caisson, but caused trouble in ramming in the concrete beneath them. Horizontal angle-braces carried round the caisson over the pockets would apparently have answered equally well, and would have left a freer space for the filling. In this connection he thought a caisson with a double shell was generally preferable, although a little more expensive to construct. It occurred to him that a slight upward slope given to the roof of the working-chamber of a caisson, from the outer walls inwards to the bases of the shafts, would considerably facilitate the final packing of the concrete beneath the roof of the chamber. He would be glad to know whether that had been tried? Coming to the superstructure, he supposed the close spacing (6 feet apart) of the cross girders carrying the roadway had been necessary in order to keep their depth within a reasonable limit, seeing that several of them occurred over the piers. Otherwise it appeared a disadvantage to attach them to the bottom boom at other points than the junctions of the main bracing, and must have entailed an appreciable increase in section of the bottom boom for meeting the consequent secondary bending stresses. Their greater number, as compared with a wider spacing, also increased the labour of connecting them with the longitudinal stringers. He ventured to doubt the efficiency of the type of track and rail-guards adopted. A very slight slackening of the fish-bolt nuts would tend to set up lateral rocking of the rails, and would cause them to bear on the timber waybeams; whilst he was of the opinion that a heavy balk of timber protected by a steel nosing was more suitable as a rail-guard than a relatively light steel section. The method of bolting the temporary towers, used during the erection, to the ends of the river spans appeared to be inferior to the adoption of a hinged tower. He was aware that in England there was a certain degree of reluctance to employ hinged members, but the excellent results obtained on the Continent and in America had fully demonstrated their value. Compared with the erection of the Niagara arch of 840 feet span, in which adjustable anchor-ties were used in connection with pin joints, the bolted bases of these towers and the manœuvring thereby involved seemed a little crude. In an important work such as this it would be of great interest and value if the calculations relating to the deflection could be placed on record. Reliable methods of calculating the exact deflection of even a simple lattice structure were rare; while the calculation of the deflection of a doubly-braced girder, in which the ties and struts were riveted at the intersections, must have been particularly intricate. One other point he wished to comment upon was the position of the bearings on the river piers. According to Fig. 13, Plate 4, the pin-bearing for the river span was 5 feet from the centre of the pier, and the Mr. Husband.

Mr. Husband. expansion-bearing of the land span about 2 feet 9 inches. The dead load alone on each bearing of the river span was 800 tons, and on each bearing of the land span 371 tons, while still heavier loading would come upon the river-span bearings during erection. Taking the case when the river span was carrying the rolling load and the land span was unloaded, the total pressure on the river-span bearing would be about 1,300 tons, and that on the land-span bearing 371 tons. The resultant of these unequal pressures would be situated about 3 feet 3 inches from the centre of the pier towards the river face, whilst with the land span loaded and river span unloaded the resultant would only move over about 1 foot 9 inches towards the pier centre. Would it not have been preferable to set back the bearings about 2 feet 6 inches farther from the river face of the pier, with a view to obtain a more nearly central loading of the pier? It appeared from the drawings that the joint between the land spans and the river spans had been placed over the centre of the pier without having regard to the large difference in span and loading. This would further have proved advantageous during the early stage of the erection, since an additional length of 2 feet 6 inches would have been available in rear of the front packing-blocks. He did not quite understand the statement that the tests indicated that under ordinary working conditions the bolts or rivets in a joint were not called upon to resist shearing stress. Unless that implied that the whole pull was resisted by friction, he would like to ask in what way the rivets were resisting? In a riveted or bolted joint, where the rivets or bolts were only loaded to a safe extent, one would scarcely expect them to show signs of shear effect or distortion, unless they had been subjected to severe reversals. Assuming a turned and fitted bolt of $1\frac{1}{16}$ inch diameter under a mean working shear of, say, 4 tons per square inch, and a reasonable coefficient of friction, it should not need very heavy hammering to remove it from its hole. The ease with which bolts were knocked out whilst under stress was scarcely a proof of the above statement.

Referring to the new Clyde bridge of the Caledonian Railway, a point in the design to which exception might be taken was the uneconomical ratio of depth to span in the main girders. It was, however, so obviously impossible, on account of the relative levels of the work, to give the girders a greater depth that this could not be regarded as a fault. It was especially satisfactory to note the care that had been taken to secure for the bridge a pleasing appearance. In a bridge of this type, which was so utilitarian in character, small blame might attach to the designers for comparative neglect of this feature. He did not wish it to be thought that he was in the least degree making any invidious comparison, but he would like to direct attention to two outstanding features which were strongly emphasized by these two bridges. Each possessed a large central span, with smaller spans on either side. The two noticeable features were, first, the long horizontal line of the upper edge of the parapet in the Clyde bridge as compared with the interrupted upper edge of the girders of the Wear bridge; and secondly, the projecting piers of the former as compared with the truncated piers in the latter bridge. There were, of course, the soundest of structural reasons for these differences. The upper booms of the river-span girders of the Wear bridge were necessarily curved on the score of economy; yet there was no doubt that the uninterrupted parapet-line of the Clyde bridge largely enhanced its appearance. Again, the carrying-up of the piers of the Clyde bridge considerably above the level of the girders characterized the structure in a very marked manner as contrasted with the Wear bridge. The presence of the overhanging footways and pipe-brackets on the latter bridge necessarily precluded any such treatment of its piers. The contour of the sky-line and the treatment of the piers constituted the two most noticeable and characteristic features in any bridge from the point of view of æsthetics. Mr. Husband.

Mr. F. E. Robertson thought the Author of the first Paper was probably correct in his remarks to the effect that the natural pressure at the bottom of a foundation did not seem to receive much consideration, although it was indeed the principal element to be considered. For example, 7 tons per square foot on sand would be an absurd pressure to attempt on a shallow foundation, but at 150 feet deep it was no more than a balance to the surrounding pressure, and it was the surcharge only which counted. Rankine's formulas for foundations, which were as good as any that could be devised, depended explicitly upon the ratio of the unit pressure from the structure to that naturally existing. In most accounts of works the facts relating to the pressures on foundations were given in a very perfunctory way, it sometimes not even being stated whether skin-friction and displacement had been deducted. The former, as a rule, was hardly worth doing, as a safe amount would be very small. With reference to the remarks on the effects of compressed air when in the clay, it might be pointed out that the proper place for the fresh-air inlet was in the working-chamber, as far from the shaft as possible: if the air did not escape through the soil it should be deliberately wasted through the lock, so that at no time should there be less air passing than 5 cubic feet Mr. Robertson.

Mr. Robertson. per man per minute at the pressure used, and too much air could not be given. To discharge the air from the compressors just below the lock meant that the fresh air was consumed in locking out while the foul air remained in the working-chamber. There was a Blue Book,[1] containing invaluable information on deep-water diving, which could be purchased for a shilling. The suggestion on p. 41 as to standardizing practice in bridge-loadings and permissible stresses was a timely one; but that was work which should be done by a Committee of The Institution. The Indian Bridge Rules were good and might be adopted with advantage. They took the form of a permissible stress referred to dead load, the figure being 8 tons per square inch. The live load was reduced to terms of dead load by an impact allowance which varied with the length loaded, from 100 per cent. on a zero span down to 60 per cent. on 200 feet span, and so on, so that the working-stress on a stringer, where the ratio of dead load was very small, would come out at about 4 tons on the static load, and the stress in the braces would vary automatically, according to the ratios of dead and live load, producing the maximum stress in the member. For compression-members a column-formula was used based on a maximum stress of 6·8 tons, however small the ratio of L/ρ might be. To standardize the loadings would be a more difficult matter, as circumstances varied so much; but a series, such as Cooper's, might well be drawn up, which should preserve the proper relation of load per foot of track to the length loaded in the case of railway-bridges. The stilt expansion-bearing used in the bridge in question had its merits, but had the fault of producing a positive movement of raising or lowering the end of the girder, which might be small in amount but was there all the time.

The Paper on the Wear bridge was of special interest to Mr. Robertson, as he happened to be engaged upon a very similar problem, the design of 360-foot double-track spans for India, some of which might have to be erected by building out. There were generally two ways and sometimes more of dealing with such a problem, and if he had arrived at different conclusions from those arrived at for the work under consideration, they might form an interesting subject for discussion. As regards the curbs of the caissons, an open frame was better than a horizontal plate at the top, as it was difficult to pack the concrete under the plates. His practice was to put a similar horizontal open frame half-way up the wedge, so that the whole, when concreted up, made a very strong girder. He preferred to make the roof of the chamber slope upwards towards the shafts, so as to render the filling

[1] Report to the Admiralty on Deep-Water Diving. London, 1907.

Mr. Robertson.

of the chamber easier. Building out was clearly the right way to deal with the river span, but with unimpeachable foundations he would have preferred to use a continuous girder. To strengthen a simple span for building out added about 8 per cent. to its weight, which 8 per cent. did not increase the capacity of the girder for its normal load. But the disposition of the metal in a continuous girder suited both the building out and the permanent loads. The girders, being so heavily loaded, seemed too shallow, and the panels were very short. He would have made them 15 feet and used sub-panels. In building out the sub-panel members could have been omitted until the girders were joined; and saving half the joints in building out was no small matter. It was not clear why the road-way cross girders were placed closer together than those of the railway, as the load on them was much lighter; and to put the lower chord under cross-bending loads was not economical. The end posts were enormous, and to make a compression-member too wide was apt to lead to poor detail. It was a very useful empirical rule that plates in compression should not be thinner than $\frac{1}{30}$, or at least $\frac{1}{40}$ of their unsupported width. Now the plates of this post must be 48 inches wide, and by this reckoning should be about $1\frac{1}{3}$ inch thick. Their thickness was not given in the Paper, and it would be interesting to know it. Mr. Robertson had used turned bolts in temporary work and in strengthening old bridges where sound rivets could not be driven: he had found them as tight as rivets, but he preferred to make them with a slight taper. It might be pointed out, though it hardly applied to these bridges where the piers were very stout, that two fixed bearings should not be placed on one pier, or one pier had to carry a double longitudinal load while another did nothing. In these days of continuous brakes the longitudinal thrust was worth noticing and so in a large span was the wind-load. It was not generally remembered that the longitudinal wind-load was about three-quarters of the side load; and when the wind blew at 45 degrees it affected a pier more strongly than a similar beam wind. He had planned for building out that the fixed bearing should be under the anchor span and the roller-bearing left free under the cantilever, so that no undue strain should be brought upon the holding-down bolts. The claim that considerable care had been taken to avoid secondary strains could only be admitted in the sense that the erectors took considerable trouble to mitigate the inconvenience caused by some features in the design, notably the enormous base of the pillar. If a knuckle-joint was good enough to carry the whole girder with the pillar on top of it, Mr. Robertson. why was it not good for the pillar itself? Then the temporary tie-plate on top of the end posts, with a kink in it, could hardly be considered a desirable arrangement. The main ties again, starting from the lower-chord gussets and passing round the wider top flange-plates, showed a considerable deviation from the straight. The thrust-block between the bottom of the end posts had a very wide bearing between surfaces, whose inclinations varied with the load. Mr. Robertson provided the bearing of such wedges with a means of swivelling, as a large bearing-area was of no use unless the pressure was fairly distributed over it. Even in the permanent structure, if Fig. 13, Plate 4, correctly represented the bearings, the centre of pressure was about 10 feet from the river edge of the pier, so that 20 feet was all the useful width the pier need have had. He believed that the points raised afforded fair subjects for discussion, without in any way detracting from the merits of a fine work brought to a successful conclusion without accident.

Mr. Stoney. Mr. E. W. Stoney, referring to the erection of the centre span of the Queen Alexandra bridge, observed that owing to its great weight the work had been very difficult, and had been carried out with great skill and accuracy. The comparative tests of joints made with turned steel bolts and with rivets were very instructive, and showed that turned bolts fitting closely in drilled holes might be used with complete confidence, their use for all temporary work being much more convenient than that of rivets. There was the additional advantage that they were easily inserted, and could also be quickly removed without racking or injury to the plates connected, which was apt to occur when temporary rivets had to be cut and drifted out.

With reference to the new Clyde bridge, it was stated that, the caisson being partly sunk into the bed of the river without any excavation having been removed, the working-chamber was usually found to be partly filled with soft slurry, such material was difficult to remove, and had to be bailed out with buckets before excavations proper could be commenced. Mr. Stoney suggested that such material might easily have been removed by any of the forms of dredgers used in India for sinking wells, cylinders, and caisson bridge-foundations, and he would like to know why such a method had not been used. The arrangements made for waterproofing the bridge-floor and for draining the ballast seemed to be complete and thorough. This matter of waterproofing and drainage was very important in relation to the preservation of girder-work from corrosion, and in older bridges it did not seem to have received that attention from designers which its importance deserved.

Mr. J. A. L. Waddell confined his remarks to a few salient points of the design of the superstructure of the new Clyde bridge. The trusses were excessively shallow, a feature apparently unavoidable on account of existing conditions. He had observed, though, both when examining English bridges and from perusal of descriptions of them, that the truss-depths used in English practice were far too small for economy of metal. The panel-lengths adopted were too short. For both economy and appearance they should be fully twice as long as in the bridge referred to. There was no good reason for employing the double system of cancellation, or the Linville or Whipple truss, because the single system of cancellation would be more economical and would present a better appearance; besides, there was a certain ambiguity of stress-distribution in the double intersection, which was objectionable. The use of flats instead of shapes for diagonal truss-members was not good practice, because it was impossible to make the components of the pairs or groups of flats act equally. Often some of such components were quite idle under the dead load, and did not become really taut until most of the live load was applied. In the case of sections that were properly attached to each other by lacing or diaphragms, this unequal distribution of stress on the component parts of the member did not exist to any appreciable extent. There were not enough counters in the trusses, and if it were not for the resistance to shear of the great chord sections the structure would certainly collapse. Mr. Waddell could not show absolute proof of the correctness of this statement, because the live and dead loads for which the trusses were designed were not stated in the Paper, but the following would indicate the justness of the claim: Mr. Waddell.

Let w denote the equivalent live load per lineal foot per truss.
w_1 „ the dead load per lineal foot per truss.
i „ the coefficient of impact.
p „ the panel length.
$W = w\,p$ denote panel live load.
$W_1 = w_1 p$ „ panel dead load.

The number of panels in the truss was twenty-four, and they were almost all of the same length, hence they would be assumed to be exactly so. The tenth panel had no counterbracing, and the maximum net shear on it would be given by the expression,

$$S_{10} = \frac{1 + 3 + 5 + 7 + 9}{24} W (1 + i) - W_1$$

$$= \frac{25}{24} W (1 + i) - W_1.$$

Mr. Waddell. As the panel live load W for a span of 200 feet was certainly greater than the panel dead load W_1, the value of the shear, S_{10}, was positive; and it could be resisted only by the sections of the top and the bottom chords. These, not being proportioned for shear, certainly must be overstressed. The shear on the ninth panel was given by the expression,

$$S_9 = \frac{2 + 4 + 6 + 8}{24} W (1 + i) - 1 \cdot 5 W_1$$
$$= \frac{5}{6} W (1 + i) - 1 \cdot 5 W_1.$$

Taking w, w_1, and i from American practice, and assuming that each truss carried a single track, also allowing liberally for the weight of the ballast, there resulted:—

$$W = w p = 6{,}250 \times 8 = 50{,}000 \text{ lbs.}$$
$$W_1 = w_1 p = 5{,}000 \times 8 = 40{,}000 \text{ lbs.}$$
$$i = 0 \cdot 7.$$
$$\therefore S_9 = \frac{5}{6} \times 50{,}000 (1 + 0 \cdot 7) - 1 \cdot 5 \times 40{,}000 = 11{,}000 \text{ lbs., nearly.}$$

That showed that in American practice, if this type of span were adopted (which it certainly would not be), the ninth and tenth panels would be counterbraced. It was true that the value of the coefficient of impact employed—namely, 70 per cent. of the live load—was probably greater than would ever actually occur, for it was intended to contain a margin of safety; nevertheless, in Mr. Waddell's opinion the upper diagonal in the ninth panel should certainly be either stiffened or crossed by a counter. However that might be, no one could deny the necessity for counterbracing the tenth panel. That such omission of necessary counterbracing was not the universal practice in England was shown by the Paper on the Queen Alexandra bridge. In it there was a riveted span of almost exactly the same length (200 feet), in which one-third of the panels were counterbraced, while in the new Clyde bridge only one-sixth of the panels were counterbraced. Such insufficiency of counterbracing reminded Mr. Waddell of an amusing incident which occurred some years ago at a highway-bridge letting in the United States. There, unfortunately, there were few legal restrictions in connection with the building of bridges, and anybody was at liberty to put up any kind of a bridge that he liked—it being understood that the owner of the structure was responsible for all damage to persons and property which might be caused by its failure. Most of the road-bridges in the United States were built by the counties under the superintendence of commissioners, who were generally ignorant of everything pertaining to bridge-design and construction. It happened, about the time referred to, that a certain irresponsible builder was making a good thing out of a patented lenticular truss bridge, which involved the use of a much smaller amount of metal than the corresponding ordinary highway-bridge. His *modus operandi* was to attend the lettings with a model of the bridge he advocated, about 4 feet long, to support it upon the edges of two chairs, and to invite two of the heaviest men in the room to sit upon it; and when it did not fail under the load thus imposed, he claimed its superiority over all other types of highway bridge, and often secured contracts thereby. On one such occasion there was present a shrewd contractor who noticed there was no counterbracing in the trusses, and when the two heavy men, who were sitting on the model, were told to rise, he exclaimed, "Hold on a minute. Let Mr. A. rise first." Mr. A. did so, the result being that the span collapsed. The box section of the bottom chords in the new Clyde bridge was objectionable, in that it would hold more or less water and would collect dirt, snow, and ice, thus subjecting the metal to rapid deterioration from rust. In approved American bridge-practice, stiff bottom chords were built open both above and below. There was apparently too much metal in the cover-plates of the chords in comparison with the webs and the flange-angles. The lateral bracing of long single $3\frac{1}{2}$-inch by $3\frac{1}{2}$-inch by $\frac{3}{8}$-inch angle-bars, with their flat inclination, was too flimsy, and was not commensurate with the magnitude and importance of the parts connected thereby. Moreover, the two rivets at the end of each angle-bar did not make a proper connection, and three was the least number that ought to be employed. Mr. Waddell.

Mr. L. Fortescue Wells did not agree with the suggestion made by the Author of the Clyde bridge Paper that all boring contractors should be officially accredited. The Author stated that it was unusual for the initial pressure of the original superincumbent earth to receive much consideration. Undoubtedly it ought to be considered when dealing with deep foundations and it was the practice of some engineers to do so. The Author entered a plea for the standardization of safe loads on foundations, but how was it possible to do this? So much depended upon local circumstances and the personal element. The depth to which it was advisable to excavate in the generality of cases became largely a matter of judgment on the part of the responsible engineer. The fact that recent practice tended towards an increase in the permissible load on various materials was worthy of attention. In earlier works engineers no doubt often erred Mr. Wells.

Mr. Wells. on the side of safety. The Table on p. 15 afforded an example of an increase in loading over what was formerly the usual practice. Ten tons per square foot used to be a very generally accepted safe working-load on blue brickwork in cement; the Author allowed 15 tons. On granite ashlar masonry the Table gave 25 tons, and on granite bearing-blocks 20 tons per square foot. At the Tower bridge 16 tons per square foot was fixed as the maximum permissible load on granite, and 10 tons per square foot was put upon blue brick-work in cement.[1] The concrete at the Clyde bridge was 5-to-1, which seemed unnecessarily rich, and 7 tons per square foot was put upon it. No mention was made of "plums" in the concrete. Were "plums" not permitted, and if that was the case, what was the objection? The occasions when it was necessary to prohibit "plums" were few, and it was difficult to understand why so many engineers still hesitated to use "plums" in concrete, especially when deposited in mass. Mr. Wells did not appreciate the restriction as to grouting brickwork, referred to in the Paper. Absolutely good brickwork might be built by laying or paving in a course on a sufficient layer of mortar and then grouting it, the process forming the layer of mortar on which to bed the succeeding course. It did not appear that any material advantage was gained by the through courses of blue brickwork in the piers as shown on the section. He wished to ask whether any settlement of the foundation was recorded before it was decided not to impose the test-load provided for in the contract. Also, what was the maximum load placed on a pile of the staging driven 12 feet into the river-bed? How was it intended to clean and paint the inside faces of the flat bars forming the tension-members of the web-bracing? In the case of a large girder within his knowledge, with tension-members of similar design, the space between the flat bars had been afterwards filled in. He thought the expediency of coating the metal with oil before painting depended on the state of the surface. When the metal could be heated and a really clean surface was dipped in oil which was allowed to form a skin, he believed it to be an excellent practice.

Mr. Matheson. Mr. Matheson, in reply, stated that the piers had been formed of a series of cylindrical pillars in accordance with the requirements of the City Corporation. It was thought that the series of pillars would give the bridge a better appearance than if the piers were continuous and solid, and in this the railway-company fully concurred. The suggestion in regard to the use of wrought iron as against the

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[1] Minutes of Proceedings Inst. C.E., vol. cxxvii, p. 36.

use of steel in bridge-building was that the cost of maintenance would be less were the former material used. It was well known that steel needed much greater attention in cleaning and painting than wrought iron, because steel corroded much more quickly. In regard to surface friction and buoyancy in connection with the foundations, there was such a thing as expediency, and, having regard to experience and true economics, the proper course had been followed in determining the safe loads. The actual depths of the several caissons had been finally fixed as the work proceeded and had been determined by the circumstances. As to the observations by Mr. Waddell, that gentleman having indicated that his criticism was based on inadequate data, it was sufficient to say, in reply, that if the bridge had to be built again it would be designed exactly as it had been designed. Mr. Matheson.

Mr. Buscarlet, in reply, referring to the general effect of the approaches of the Queen Alexandra bridge, observed that the idea of carrying the roadway of the approach-viaducts on columns instead of arches had never been entertained. Owing to the height of the roadway, especially on the north side of the river, such a scheme would not have been, in his view, in keeping with the massiveness of the railway-approaches and river-piers. It was, however, a matter of opinion. The columns supporting the railway of the northern approach were placed in rows of three, one on either side of the roadway, and the others following the curve on the west side of the railway, with additional ones where required. The railway cross girders were continued over the roadway and supported a flooring similar to that of the railway. With reference to the close spacing of the girders carrying the roadway, it was questionable whether the placing of them at greater distances apart would have diminished appreciably the weight of the whole roadway (cross girders and flooring). Apart from this, the choice of depth had been influenced by the headway required over the river and by the necessity of keeping the railway at as low a level as possible. Also, by placing the hangers between the verticals, a clear space for fixing them was obtained. Referring to Mr. Robertson's remarks, the spacing of the railway cross girders was governed by the distance apart of the verticals. The thickness of the plates of the end posts was $\frac{1}{2}$ inch. Two expansion-bearings (not fixed bearings) were placed, as shown in Fig. 3, Plate 3, on pier No. 2. Mr. Buscarlet.

Mr. Hunter, in reply, stated that the broad base for the temporary towers had been adopted to facilitate their erection and to suit the construction of the end posts of the main girders. The slacking back of some of the nuts on the bolts in the bases, as Mr. Hunter.

Mr. Hunter. described in the Paper, had given all the freedom of a hinged joint without its disadvantages. The methods by which the deflections had been computed and the allowances which had been made for the effect of the riveted connections and cover-plates were described in the Paper. The deflection of the cantilever main girder supported on the pier and by the inclined temporary ties (assuming these two points to be in a level line), were first computed. The thrust along the bottom boom of the main girder due to the angle of the inclined ties was included in the forces producing this deformation. The droop of the main girder at the point of attachment of the temporary ties was then computed and was due to the elongation of the front ties and the anchor-ties, and to the shortening of the towers, the end posts and the land spans. The main girder was then considered as revolving around the pin bearing on the pier, and the total deflection of the main girder was determined for any point in the span. Mr. Husband's interpretation of the reference to the stresses in riveted joints under working conditions was correct. Mr. Hunter was pleased to know that Mr. Robertson's experience with turned bolts in bridgework was in agreement with the conclusions given in the Paper. He did not agree, however, that it was necessary to have a taper on the bolts. To ensure a uniform bearing, if any slip to cause bearing stress occurred, the holes should be rimered parallel, and parallel bolts used which had a hard driving fit. The diameter of the bolts should be larger in the body than in the screwed portion, in order to avoid damage to the threads in driving them home. If tapered bolts were used they should be driven in alternately from opposite sides of the joint to ensure that one cover was not taking more stress than the other cover. In regard to the position of the bearings on the cantilever and fixed piers, it was desirable for erection purposes to have fixed bearings on both cantilever piers, but provision must be made on one of these piers to convert the bearings under both land and river spans into an expansion-bearing immediately the junction was made in the centre of the cantilever span, and for these bearings to remain so until the temporary work was removed. An important point to remember, and one which had been demonstrated in the erection of the Wear bridge, was to see that before beginning the erection of the cantilever span the locked rocker-bearings were set sufficiently over towards the cantilever span to allow for the extension in length of the bottom boom of the main girders when they were relieved from the support of the temporary ties and became an independent span. The temporary tie-plate at the top of the end posts was an additional tie for emergency purposes. The set in the tie-plate came immediately below the base of the towers, so Mr. Hunter. that the objection raised to the set in it was not a serious matter. The long length of the inclined temporary ties rendered the slight set in them required to pass the top booms of little moment. The great strength and stiffness of the end posts of the main girders ensured a fairly uniform distribution of the thrust over the section of the bottom booms of the main girders due to any variation of the centre of pressure on the thrust-blocks. The precaution was taken, however, to stiffen and brace the upper edges of the vertical web-plates of the boom to resist any increased pressure. With light end posts a swivelling arrangement would have been necessary to keep the centre of pressure in one line. In addition to swivelling motion, vertical sliding must also be provided for.

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12 April, 1910.

ANTHONY GEORGE LYSTER, Vice-President,
in the Chair.

The discussion upon the Papers by Messrs. Matheson and Buscarlet and Hunter, on the new bridges over the Clyde and the Wear, occupied the evening.
