Mr. Hayter. sunk to the ground-line or thereabouts; and when that was reached a length of the same diameter could be added temporarily until the full depth was attained. The ground outside could then be sloped away as far as necessary, the temporary length withdrawn, and the conical piece with the upper portion of the smaller diameter could be permanently fixed. If this plan had not been followed, as he had supposed it would have been, it was evident that the cylinders need only be contracted when sinking the last few feet, which would not be a matter of much moment. At all events he should have been sorry not to have reduced the diameter of the upper portion. To have omitted to do so would have added to the cost of the work unnecessarily. Besides, if the larger diameter of 12 feet had been continued to the underside of the girders, which was at most only about 20 feet above the bed of the river, the piers would have been unsightly, as their size would have been out of all proportion to their visible height and to the span of the openings. Mr. Berkley had alluded to cast-iron cylinders cracking during sinking. Mr. Hayter had referred to such occurrences in his opening remarks, stating that he had provided against the contingency by making the bottom length of wrought-iron strong enough to take up any strain that might be superinduced during the process of sinking. In conclusion he would remark that the Chittravati bridge was one of the least costly of the kind ever erected under like conditions, and at the same time the official and other tests proved that it was a structure of proper rigidity. He would only add that he was sure Mr. Stoney would be well satisfied with the favourable reception accorded to his Paper.

Correspondence.

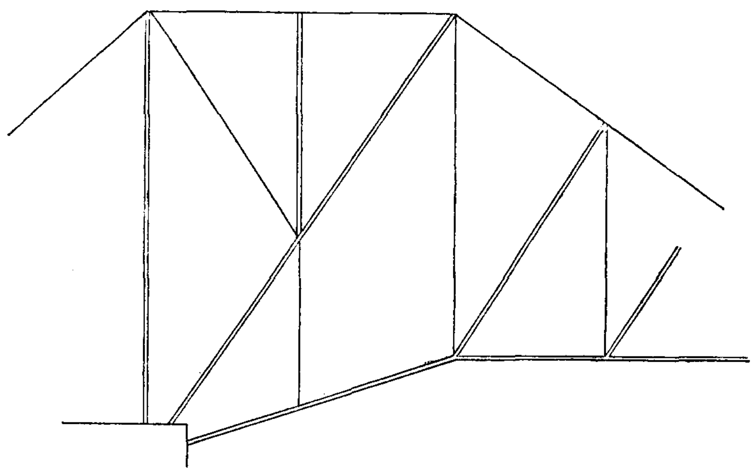
Mr. Bouscaren. Mr. G. Bouscaren said that with regard to the cantilever span of the Sukkur bridge over the Indus, Mr. Robertson's Paper was specially interesting to American Engineers, as illustrating some points of difference between the methods of bridge-building in vogue in England and America. The first question which the designer of a bridge must answer before the general features and details of his plan could be determined was, "How was the structure to be erected?" In the case of an 820-foot cantilever span, as at Sukkur, this question would very probably have been answered in America "by building out with a traveller;" and one of the principal reasons for this was, that large structures were now

Mr. Bouscaren. rarely erected by the contracting bridge companies, who preferred to confine themselves to the shop work, and sublet the erection to other parties who made a special business of that class of work, and were well equipped and trained to do it in that particular way. The result would probably have been a very different structure, especially as to details, from that designed by Sir A. M. Rendel. The commercial necessity of bending to the consideration of cheapness, arising from a keen competition, did not, perhaps, obtain in England to the same extent as in America, and left more latitude to the originality of the designer. The use of wire-rope carriers, although not a new feature in bridge erection, had been very ingeniously applied in this case, and seemed to have answered the purpose admirably, as no accident or mishap was recorded in Mr. Robertson's account of the work; but the time actually consumed in the erection of the span, from November 1887 to February 1889, seemed long, as compared with American practice, even after making all due allowance for the importance of the structure. Mention was made incidentally in Mr. Robertson's Paper of the large amount of drift carried by the river at certain seasons of the year, and this naturally called attention to the inclined booms at the abutment ends of the cantilevers, and the close proximity of the lower ends to the water suggested a liability to injury from drift, as well as danger to the crafts coming under the bridge at high water. It was a matter of regret to him that with all the valuable information given in Mr. Robertson's Paper, a little more had not been added with reference to the general specifications for the superstructure, as for instance, the live load designed to be carried by the bridge, the grade of steel used, and the limiting stresses per square inch allowed for the different members of the span. Such data would have been of great interest and value in comparing English and American practice at the present time.

The local conditions at the Chittravati bridge seemed to have been all that could be desired, and full advantage appeared to have been taken of them in the execution of the work. It was not clear, however, from Mr. Stoney's interesting account of the foundation work, why the inside curbing of masonry, which was found so useful in assisting by its weight in the sinking of the three cylinders under the north abutment, was not used as well for the cylinder-piers. The comparatively great cost of the pneumatic process as applied to some of the cylinder-piers for the removal of the large boulders appeared to have been due chiefly to the insufficiency of the plant; and with adequate machinery it seemed probable that time and expense could have been saved by a more liberal application of this process in lieu of divers. In cylinder-foundations, where rock and boulders were liable to be encountered, he would give preference to wrought-iron over cast-iron for the cylinder-shells. With an inside curbing of masonry, no danger of deformation in the process of sinking from lack of rigidity need be apprehended, and the wrought-iron was better adapted to resist the air-pressure and the vibrations from the small blasts used in removing large boulders and levelling the bed-rock. Mr. Bouscaren.

Mr. THEODORE COOPER remarked that the span of the Lansdowne bridge alone would make it a notable structure. The peculiar Mr. Cooper.

Fig. 3.



skeleton of its trusses, the extraordinary act of putting up each of its large cantilevers at the makers' yard before shipment, the difficulty of erecting the structure, and the statement of its cost, rendered it especially interesting to American bridge engineers. Facility and cost of erection did not seem to have received any consideration in the selection of the proportions of the skeleton. Without endorsing the general form of truss adopted for this bridge, he thought that a very slight change in the triangulation would have been a great improvement, especially when the erection was considered. Mr. Robertson's description of the steps necessary in order to connect together the first panel of the cantilever, emphasized very strongly the faulty division of the truss at this point. Had it been subdivided by a vertical line, Mr. Cooper. making two panels of 61 feet 6 inches, and a diagonal tension-member extending from the top of the pillars to the centre of strut No. III, as in *Fig. 3*, it could all have been erected and made self-sustaining by means of a crane or derrick of only moderate reach. The reduction in weight by thus sub-dividing the present long members, and therefore lessening the bending strains, would have exceeded the additional material needed for the new members. Other modifications could have been made, having in view the same object. The absence in this design of features considered so essential for economy and facility of manufacture and erection by the American bridge-builder, rendered an examination of the statement of cost very instructive. The total weight of iron in the trusses was given as 3,316 tons, and the cost of the ironwork was Rs.17,01,000, or £120,487, taking the Author's rate for a rupee. The cost of the erection was Rs.5,61,223, or £39,753, omitting photographs and labour for painting. These figures made the cost of ironwork £36 6*s.* 8*d.* per ton; cost of erection £11 19*s.* 9*d.* per ton, giving a total for ironwork erected of £48 6*s.* 5*d.* In addition, it was difficult to exactly apportion the other items, such as charges for quarters, workshops, boats, plant, and contingencies, which would probably bring up the above amount to £50 per ton in the finished bridge. Presumably the cost of the ironwork alone, as above given, included that of the preliminary erection at the makers' works. Assuming that this, together with the taking down again, would amount to as much as the second erection, nearly half the money was expended in this way. As the cost of erecting such a bridge should not have exceeded £6 per ton, and as, according to the American practice, accuracy of length of the parts and correctness of fitting of the connections would have been attained without the preliminary erection, American bridge-builders would gladly have discounted the actual cost of the ironwork erected to the extent of £18 per ton at least.

Mr. Fidler. Mr. T. CLAXTON FIDLER said the two bridges described in these Papers could hardly be compared. They presented a wide contrast in the lines of their design, and if possible a still wider one in the means adopted for their erection. At Chittravati the engineer boldly ventured down upon the bed of the stream, and making the best use of the dry season, succeeded in erecting his long line of girders by a method of the greatest simplicity, and at the lowest possible cost. But at Sukkur the erection of the cantilevers over the rapids in that confined gorge of the Indus presented difficulties of a very different order, which were surmounted by the employment of the

Mr. Fidler.

ingenious system of wire-rope transport described in Mr. Robertson's Paper. Of course the Sukkur bridge was not the first that had been erected by a method of overhead suspension; but in this example the appliances seemed to have been worked out, in all their details, with great ingenuity, and being admirably adapted for the difficulties of the situation, they appeared to have been employed with perfect success. In connection with this wire-rope rigging, there were one or two points on which some further information would perhaps be desirable. Whenever a member of the web-bracing was sent out for erection, it had to be suspended in a transversely battering direction, and the head of the piece was held in its true position by a rope or pair of ropes, hanging in the vertical plane of the top member; but to give it the requisite lateral spread at the foot the heel-rope must apparently have been worked from some sort of yard-arm, or spinnaker-boom, rigged out laterally from the gallows-frame. Something of this kind was incidentally referred to in the Paper, but the drawings did not show this spinnaker-boom, and it would be interesting to know its length, and how it was rigged and worked. It appeared also that, in a general way, the pieces were picked up from barges moored out in the river, but sometimes the barges could not be used, and on such occasions the Author did not explain by what course the pieces were taken out into position, and how they were steered up aloft so as to avoid fouling with the existing work. Another remarkable feature was the preliminary erection of the cantilevers on staging in the makers' yard. It would probably occur to most engineers that the construction of this great timber scaffold must have been attended with an expense which seemed disproportioned to the object in view, if that object was nothing more than to present the parts together so as to secure their accurate fitting; although the adjustment of such members, meeting at varying angles of transverse inclination, might very likely have been a complex matter. It was obvious, however, that such a proceeding would greatly facilitate the erection in mid-air by this wire-rope system of suspension; and perhaps the timber stage at Millwall, and the wire-rope rigging at Sukkur ought to be considered as complementary parts of the same scheme. It would be interesting to know how far they were so regarded by the engineers engaged in the work. Apart from the method of its erection, the Sukkur bridge presented some remarkable features in its design, which might be an interesting subject for discussion if the materials were at hand. But the Paper was concerned with the erection rather than with the

Mr. Fidler. bridge itself. The drawings showed the anchorage only in diagrammatic outline, and did not give the sectional area of any of the steel members; while no information was afforded as to the live load or the contemplated wind-pressure, nor as to the deflection of the bridge under these forces. It was evident, however, that the structure differed from the ordinary form of cantilever-bridges. It was not a balanced cantilever anchored vertically downwards at the tail end, but might perhaps be described as a single-armed cantilever, strutted against the abutment at the foot, while the head was held back by an inclined back-stay and anchorage, like those of a suspension bridge. The adoption of this form was evidently attended with certain consequences. The contraction and expansion of the backstay must cause the pillar to rock upon its bearings, and the cantilevers to rise and fall at the outer end. These movements would be produced by change of temperature, and also by the imposition and removal of the live load. So far as they were due to change of temperature they would be entirely avoided in the ordinary form of balanced cantilever; but, on the other hand, the horizontal movement at the top of the pillar, and the consequent drooping of the outer end, due to the imposition of the live load, would be less in this tied cantilever than in the ordinary balanced form. The maximum movement, from a condition of no load and cold temperature to a condition of full load and hot temperature, would of course be the sum of the two separate effects; but unfortunately the movement due to load could not be calculated in the absence of the necessary data as to the weight and as to the sectional area of the members; though it appeared probable that the maximum movement in this design was on the whole greater than it would be in a balanced cantilever. It was, no doubt, provided for in the calculated fibre-stress arising in the slender ankles of the pillar and main strut. The comparative anatomy of bridges suggested also the question of the cost of this inclined anchorage, as compared with that of a vertical anchorage and a steel strut, by which it might conceivably have been replaced. The question, however, would be governed by local considerations, and the presence in this case of the natural rock at a level considerably higher than the abutment, might have suggested the method here adopted for counterbalancing the cantilever.

Mr. Gaudard. Mr. JULES GAUDARD recalled the fact that the Fribourg suspension-bridge had been erected in 1832–34 by the French engineer Chaley, across a span which was equal to that at Rori, and which was at the time the widest that had ever been bridged in any country.

Mr. Gaudard.

At Niagara and at Brooklyn, Roebling appeared to confirm the idea that suspension-cables alone offered such a combination of lightness and tenacity as could be trusted for spans of such size; but a method of constructing rigid arched-ribs was making its way, in which wire cables or back-stays were used only as a temporary support for the permanent structure during the successive phases of its erection. In this way the cast-iron arch of 230 feet span, over El Cinca in Spain, was built in 1866 by Messrs. Schneider and Co. of Creusot without any scaffolding; and by the same means, in 1873, Eads erected the three great steel arches at St. Louis, which were again surpassed in width by the Garabit arch of 541 feet, and by the arches of Maria Pia and of Dom Luis at Oporto, which were executed by Messrs. Eiffel and Seyrig, and all of which were erected without any staging, by the aid of wire-rope stays. At St. Louis the temporary suspenders were accompanied by trestles of timber which abutted upon the piers, and flanked them on either side in the manner of cantilevers. But if the cantilevers were capable of sustaining the weight of the permanent bridge, why could they not take its place in sustaining the weight of the train? Such was the consideration which had guided Messrs. Fowler and Baker in spanning the Firth of Forth. Like the shooting stem of a plant, the cantilever sustained by its own strength the successive elements which it assimilated in its progressive growth; so that the temporary supports, instead of having to carry ultimately the whole weight of a semi-arch, had only to carry the fractional members of the structure, which themselves were gradually built out by the aid of movable platforms. From the day when this method of building not only chimney-shafts and light-house towers but also inclined members, was learnt, the art of erecting colossal structures made a new and rapid advance. The Rori girder was a modern witness of this fact, and it remained only to record in fitting language the consummate ability of the engineers, and the coolness and bravery of the workmen of all ranks who had co-operated in its erection. Some slight criticism might, however, be offered on æsthetic grounds. The Forth bridge had been objected to on account of its inelegant appearance, which was like that of a huge mass of scaffolding; although nothing could be more rational than its general outline, which recalled the figure of a diagram of bending moments in a continuous girder, with its parts grouped symmetrically about the supports. But it must be avowed that the appearance presented by the Sukkur bridge seemed far from being an artistic improvement. The logic of it, certainly, was conspicuous enough. Every stage

Mr. Gaudard. in the execution had been wisely calculated. The abutment offering a point of support for the thrust of the lower boom, the latter had been made use of to sustain the foot of the vertical member IV (Fig. 1, Plate 4), just as a pier would support it; hence this vertical might be treated as forming a counterpart to the pillar A, with which it was connected by the horizontal member AB; while from the summits A and B reached in each direction the back-stay or "guy," and the tie BE, sloping with symmetrical inclinations. But it was precisely this false symmetry about a wrong axis that had something hipped and limping about it. The jump or sudden change of height which occurred at the end of the central span, separating it from the pointed nose of the cantilever, expressed the articulation of the system; but the defect of level was not without an injurious effect upon the appearance. It was not clearly apparent why the struts of the web-bracing should change the direction of their inclination on each side of the vertical II; and moreover, the swelled form of the great struts was perhaps a little exaggerated. It would be interesting to discuss the comparative merits of this connecting-rod form, and of the tubular form adopted at the Forth bridge, which was more consolidated, well able to resist compressive stress, and lent itself easily to the progressive shifting of the movable platforms, but presented on the other hand the inconvenience of a complicated intersection at the joints of the framework. If, then, cantilever bridges aspired to better æsthetic conditions, it appeared that for them, beauty could hardly consist simply in "the splendour of truth;" but that, as architects confessed when they built false windows, art sometimes demanded a little dissimulation.

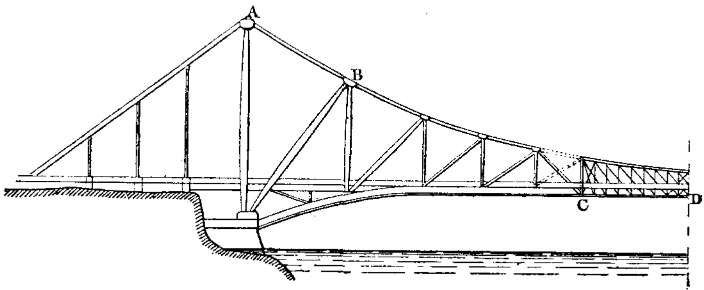
Mr. Gaudard then referred to a preliminary design prepared by Messrs. Bartissol and Seyrig for a bridge at Lisbon.[1] Proposing a system of cantilevers, the Authors disguised the break at the articulations under the elegant appearance of arches with a continuous elliptical curvature, sacrificing the logical form of the central girder, which was made to decrease in height towards the centre instead of giving it the rational bowstring form. In structures placed above the bridge-platform, it was at least possible, in default of the arch, to imitate the curvature of a suspension-bridge. The girder at Rori, for example, might be modified as in *Fig. 4*. Referring to the Chittravati bridge, Mr. Gaudard remarked that it was a very substantial work, the features of which

[1] Paris. imp. Barré, 1889.

Mr. Gaudard.

had doubtless been dictated by local conditions connected with the transport of materials and the employment of native labour. The old bridge having been undermined, the constructors of the new one had desired, at any cost, to sink the foundations down to the solid rock. With the object of employing, as far as possible, an economical construction in concrete, they had recourse to cast-iron cylinders, which were mostly sunk with open tops to facilitate the dredging, although they had not neglected to avail themselves of subsidiary appliances, such as pulsometer pumps, diving operations, and the use of compressed air. But although piers founded upon the rock might well justify the adoption of continuous girders, they had employed independent girders throughout, and this form had probably been selected in consequence of the separate erection of each span upon the bed of the river. Subject to these considerations, Mr. Gaudard went on to indicate the points

Fig. 4.



in which this work seemed to differ from the prevailing tendencies of continental practice. Since the execution, in 1859, of the foundations of the bridge at Kehl, by Vuignier and Fleur-Saint-Denis, masonry piers in single blocks had generally been preferred to multiple columns with cast-iron casing—a preference which was even more strongly grounded in the case of a narrow single-line bridge. The pier could be kept plumb in the process of sinking with great certainty; and if any deviation took place, it was of less importance when the constructor was relieved of the necessity of fitting bracings, or connections between the columns; the progressive building of the masonry, *pari passu* with the sinking, dispensed with the employment of auxiliary loads—at least in the case of pneumatic sinking, which required only small shafts; and lastly, there was the endeavour to reduce the employment of metal in the piers, even though it might be attended with certain risks of settlement or cracking when the upper part of the

Mr. Gaudard. masonry was hung up in some tenacious bed of material while the lower part descended. It might be presumed that the piers of the old bridge rested in the sand, without reaching either the clay or the boulders; at the depth where the boulders were buried, they would seem to be safe from undermining, and to constitute a sort of coarse concrete nearly as good as that concrete by which they were replaced. If, when the cylinders had been sunk more than 50 feet below the river-bed, a material was met with that was difficult to excavate, the fact was an indication that the sinking might be stopped with perfect safety. Probably the screw-piles which had held good in the old work, had reached the point of refusal at the first large stones that were met with; and the conclusion that it would have sufficed to stop sinking on meeting the boulders, seemed to be again confirmed by the weight of rails employed to force the descent of the cylinders. For example, pier No. 11, when empty and undermined at the base, carried 444 tons of rails (222 on each cylinder), while the pier when completed and properly bedded, would only have to carry 160 tons of dead load, and 177 tons of test-load, or 337 tons altogether. The frictional resistance varied with the depth and with the lateral pressure exerted by the soil. If the soil was supposed to be without cohesion at all depths, so that the total pressure increased as the square of the height, the idea of a pressure per square foot, the variations of which were shown in Table II, might be replaced by an abstract number or a coefficient of friction, constant for any given material. As it was difficult to estimate correctly the pressure of earth, it had been proposed to take as a basis of comparison the simple pressure of water. Applying this idea to the friction of 2·13 cwts. per square foot, given in Table II, and referring to the depths, of which the mean was 41 feet, the following results were obtained:—Friction for this depth $= 2{\cdot}13 \times 41$; corresponding imaginary pressure of water $= 0{\cdot}2787 \times 41^2$; ratio $= 0{\cdot}19$. Table I, with a mean friction of 2·71 cwts. for a mean depth of 55 feet, gave a ratio of 0·18, agreeing very well. In the case of metallic caissons sunk in gravel or sand the value of this coefficient had been found to be from 0·4 to 0·6; but its value would be greatly influenced by the unknown degree of the cohesion of the soil in the deep beds. The ruptures occasioned by the explosion of dynamite might be referable to the want of space, for in the vast caissons at Brooklyn powder was employed without danger. However, cylinders of 8 feet 2 inches had been quoted at Palma del Rio on the Guadalquiver, where explosives were used to produce a kind of earthquake, with a view of prompting the descent. The last observation he would make with reference to the Chittravati bridge was that with foundations which were at once sufficiently costly, and abundantly safe, European practice would certainly have increased the width of the spans and connected the girders. The bridge at Bordeaux had piers of the same calibre—that was to say, they consisted of double columns, 12 feet in diameter, but without tapering at the top—which carried girders of 254 feet span, and that with a double line of railway. The foundations had been carried down to the gravel (by compressed air) at a mean depth less than that at Chittravati, viz., 25 to 56 feet below the bed; while the total height of the columns was from 78 to 87 feet. It did not appear that the very convenient process of erection employed at Chittravati should constitute an imperative reason for abandoning the continuity of the girders. In an analogous case, Robert Stephenson united the tubular girders of the Britannia bridge over the piers, although their acquired deflection rendered the continuity imperfect. But now that the method of accurately calculating the deformations was known, the theoretical conditions admitted of being more perfectly realized. When a new girder in the series had been lifted into place, it was only necessary to give to its forward extremity a super-elevation, so as to make the rear extremity prolong tangentially the deflected line of the preceding girder; and then, when the riveting was done, the letting down of the forward extremity upon the bed-plate would procure for the preceding span the relief afforded by continuity in respect of the dead load. Mr. Gaudard.

Mr. Thomas Gillott noticed with approval in the Chittravati bridge that the plates and angles in the boom-joints, where separated for shipment, were broken with splices, and not square across, as was often done. This involved more riveting on the site and greater risk of damage in transit, but made sounder work; and he asked the Author whether any injuries occurred during conveyance, such as would cause him to alter the breaks of the plates and angles, had such a design to be repeated. The number of rivets in each span requiring to be put in on the site (11,336) was equal to seventy-seven rivets per ton of work, which was somewhat high, seeing that the bridge had not a plated floor; and he would ask the Author what percentage of loose rivets put in by the native workmen had to be cut out on inspection. Some of the important ones connecting the cross-girder ends through one boom web, gusset, and vertical strut, appeared as though they would have to be knocked down single-handed (*i.e.*, by one man), and if the points were heated in a fire it would not be easy to Mr. Gillott.

Mr. Gillott. get the holes well filled under the rivet-heads. This led him to point out the advantage of the portable compressed-air riveters which he had recently adopted, and by which the greater part of the work could have been closed. A portable furnace with an air-blast would heat 200 or 250 $\frac{3}{4}$-inch or $\frac{7}{8}$-inch rivets of average length per hour, with a consumption of about $1\frac{1}{2}$ gallon of creasote oil, costing in this country *2d.* to *2½d.* per gallon; and as they were heated throughout their length, the holes were far better filled than when only the points were made hot, as was done when an ordinary fire was used. As an air-pressure of 45 lbs. per square inch would suffice for $\frac{7}{8}$-inch iron rivets, there was no trouble with leaky joints, or burst hose-pipes, and the work of one of these machines was equal to that of three sets of hand-riveters. This was an important item in the erection of bridges, and he would direct attention to the advisability of the designs being prepared so that power-riveters could be used as much as possible.

Mr. Hogg. Mr. C. P. Hogg remarked that with reference to the information given by Mr. Stoney as to surface-friction in sinking the cylinders of the new Chittravati bridge, it might be observed that the friction per square foot of imbedded surface depended not only on the nature of the strata, but even to a greater extent on whether the cylinders were exactly vertical, for the greater the deviation from the vertical, the greater would be the friction per square foot. In the Alloa railway bridge across the Forth, erected in 1882–84, there was nearly 2,000 lineal feet of cylinder sinking. The cylinders were 5 feet, 6 feet, and in some piers 8 feet in diameter. During the progress of the works several good opportunities occurred for accurately observing the surface-friction, and the engineers, Messrs. Crouch and Hogg, MM. Inst. C.E., found it to vary from 2 cwts. to 5 cwts. per square foot, the higher rates being observed when the cylinder was out of the vertical, or on resuming work after the operations had been suspended for several weeks. One of the 8-foot cylinders was sunk 74 feet below the river-bed through the following strata:—

	Ft.	Ins.
Silt and sand	2	0
Muddy sand, clay, and stones	14	0
Sandy mud, and stones	14	0
Running sand, mud, and stones	17	0
Hard sand, stones, and clay	2	0
Blown sand	14	0
Clean sand	9	0
Hard gravel, sand, and clay	2	0
Total	74	0

At the finish, the surface-friction was 2·37 cwts. per square foot of imbedded surface. In the observations made at the Alloa bridge there was nothing to show that, under similar circumstances, the surface friction increased per square foot as the depth increased. This had been quite confirmed by observations made during the sinking of the caissons of the Dalmarnock bridge, just completed by Messrs. Crouch and Hogg across the Clyde at Glasgow. The caissons for the piers of Dalmarnock bridge were constructed of wrought-iron, and were sunk by the pneumatic process from 50 to 55 feet, through fine muddy clay and sandy mud. They were of an oblong form, with parallel sides and semi-circular ends, the dimensions at the cutting edge being, length 63 feet, and width 9 feet, and at 56 feet above the cutting edge, length 62 feet 3 inches, and width 8 feet 3 inches. On account of the large area of imbedded surface the observations were of considerable value. The results were given in the accompanying Table, and might be compared with those on p. 290 of vol. li. of the Minutes of Proceedings. The net sinking-weight in the Table was the weight of the caisson, concrete, air-locks, &c., minus the lifting force due to the air-pressure in the working chamber at the moment the caisson began to sink. The values of the surface-friction were probably somewhat high as the caissons were slightly twisted. Mr. Hogg.

TABLE OF SURFACE-FRICTION AS DEDUCED FROM OBSERVATIONS MADE DURING THE SINKING OF THE CAISSONS OF DALMARNOCK BRIDGE, GLASGOW, BY THE PNEUMATIC PROCESS.

Caisson.	Depth of the Cutting-Edge below the River-Bed.		Area of the Imbedded Surface of Caisson.	Net Sinking-Weight = Weight of the Caisson, Concrete, &c., minus the Lifting Force due to Air-Pressure.	Surface-Friction per Square Foot of the Imbedded Surface of the Caisson.
	Feet.	Ins.	Square Feet.	Cwts.	Cwts.
No. 1	38	9	5,251	18,974	3·61
	46	6	6,301	24,674	3·92
	49	5	6,684	25,754	3·85
	53	5	7,211	25,754	3·57
No. 2	47	1	6,380	22,594	3·54
	53	0	7,155	24,640	3·44
	54	1	7,301	24,640	3·37

Mr. Macdonald. Mr. CHARLES MACDONALD observed that the Paper presented by Mr. Robertson, on the Lansdowne bridge, being confined to a description of the mode of erection, with but general reference to the design, must necessarily narrow the range of discussion within limits which were scarcely adequate to the importance of the subject. The problem was to construct a bridge across a clear opening of 790 feet, without the use of temporary supports from the river-bed; and as a matter of course, with due regard to the cardinal principle that the best engineering was that which most fully answered its purpose at the least cost. The cantilever type of superstructure was doubtless selected by reason of the fear that the false works required to sustain a simple girder during erection would be carried away by drift. At this distance, and without sufficient knowledge of the river in question, it was impossible to say whether a different design would not have been advisable. If there were any periods of quiet water on the Indus, and if such a period (which need not exceed eight weeks) could have been relied upon at any given season of the year, it was safe to say that a plain truss of 800 feet span, between the centres of the end supports, could have been substituted for the present design at a greatly reduced cost. Assuming, however, that the cantilever type was the most available, it was not probable that the particular arrangement adopted at Sukkur would be repeated, from motives of economy, at least. Referring to the table of weights it would be seen that after deducting roadway and rails, the structure weighed 3,220 tons, distributed as follows: from anchor to centre of pillar, 420 tons, each side; from centre of pillar to centre girder, 1,062 tons, each side; centre girder, 256 tons. From this it appeared that the weight per lineal foot of the several divisions was as follows:—

$$\text{Anchor arms, } \frac{420 \text{ tons}}{247\cdot77 \text{ feet}},\ 1\cdot7 \text{ ton, or } 3{,}808 \text{ lbs.}$$

$$\text{Cantilever arms, } \frac{1{,}062 \text{ tons}}{310 \text{ feet}},\ 3\cdot42 \text{ tons, or } 7{,}660 \text{ lbs.}$$

$$\text{Centre girder, } \frac{256 \text{ tons}}{200 \text{ feet}},\ 1\cdot28 \text{ ton, or } 2{,}867 \text{ lbs.}$$

The disparity between these figures indicated an unscientific division of lengths, as between the central span and the cantilever arms. An increase of the length of the centre span would undoubtedly result in a decrease of the total weight of the bridge. The most striking feature, to an American engineer, in this table

Mr. Macdonald.

of weights, was the excess of material required over what would be considered the best practice in the United States. It was quite within bounds to assert that a saving of at least 30 per cent. might have been made by a re-arrangement of the general proportions and modification of details so as to permit of economical erection; and this, without in the least impairing the strength or durability of the structure. It was to be regretted that the cost of the Sukkur Channel was allowed to appear in this connection, as the tendency was to confuse the statement of cost of the Rori Channel, which was the only one calling for special remark. The item marked ironwork for the Rori Channel was put down at Rs.17,01,000, which, converted into pounds sterling per ton (assuming the value of the rupee to be 1*s.* 5*d.*, and the weight 3,316 tons) became £36 6*s.* 8*d.* Probably a considerable part of this cost was to be accounted for in the expense of assembling "the entire steel-work upon a timber scaffold in the makers' yard before shipment," as reported by the Author. A bridge which was properly designed, and faithfully inspected during its manufacture at the shops, was certain to come together on the ground; and there could be no excuse for compelling the purchaser to pay the extra expense of shop erection, unless it was to shift the responsibility of accuracy from the shoulders of the engineer to those of the manufacturer. Hundreds of thousands of tons of bridge-work were erected in America every year; and, if a single span had been assembled at the shops during the past decade, it had been the exception to a universal rule. The methods pursued in erection appeared to have been judicious and economical, considering the difficulties inherent in the design. The cantilever presented most favourable conditions for the men in the field, when the details were so arranged as to permit of movable derricks. In this case, if the original design had involved a permanent tie from the top of the pillar to the middle of strut III, with a suspender from this intersection to the floor, and a strut upwards to the horizontal tie, a great saving in weight and in cost of erection would have ensued. The erection appeared to have cost £12 3*s.* 6*d.* per ton, irrespective of some small items of general expense. This was equivalent to 2·63 cents per pound, American money; and was about double what similar work was done for in that country. It was scarcely reasonable, however, to criticize the cost of work when men were subjected to a normal temperature of 100 degrees, with a maximum of 180 degrees in the sun. The wonder was that so much was accomplished, under such unfavourable conditions.

Mr. Macdonald. The delays occasioned by the removal of boulders from many of the cylinders would seem to indicate that the pneumatic method of sinking might have been used to advantage in the Chittravati bridge. It was a matter of surprise to note the high cost of what little was done by this process as compared with dredging and diving. In America, in all western rivers where the bottom was liable to scour, it was the custom to sink masonry piers by compressed-air, the caisson containing the air-chamber being constructed of timber. By this method it was comparatively an easy matter to remove obstructions, and when the element of time was considered the total cost of the work was greatly reduced. In the case of the Indian rivers, where masonry was not required to resist ice, it was a good practice to build cylinder piers; but it would be quite possible to connect the cylinders in pairs by an air-chamber at the bottom, supplying the weight for sinking by piling loose stone upon the space between them, and if necessary carrying up a concrete lining inside. By this method the boulders which caused so much delay in piers Nos. 10 to 14, could have been taken up through the air-locks at moderate cost, and nearly all the expense attending the loading and unloading of the cylinders would have been saved. The superstructure of the Chittravati bridge was said to be of the Murphy-Whipple type. This was rather a strained application of the term. Messrs. Murphy and Whipple were pioneers in developing a system of bridge construction specially adapted to rapid and economic erection, in which the principal members were connected by pins, and the only field-rivets required were of secondary importance. A truss, such as either of these gentlemen would have designed for similar spans to those at Chittravati, could have been coupled up and made self-sustaining in a few hours, and the completion in every detail assured within three days. In the bridge described in this Paper there were no less than eleven thousand three hundred and thirty-six rivets to be driven in each span before it was ready to be lifted into place; and this fact should make an engineer hesitate before selecting a type of construction which involved so many chances of imperfect work in the field, to say nothing of the increased risk of loss from flood, owing to the prolonged exposure upon the supports; or of the extra cost of doing work by native labour which might have been done to better advantage at the shops. The amount of metal put into bridge trusses in India, judging by the examples described in recent professional papers, could not be accounted for upon any rules of economic proportion with which American engineers were familiar. The train-load could not be heavier than that in use in the United States, and the factor of safety was substantially the same. Why, then, should a span of 140 feet over all weigh 146 tons 12 cwt. in India, while a span of the same length, strength, and durability, weighed but 80 tons in America? It would be of interest to know whether the equation of cost between the piers and superstructure was such as to give minimum results for the completed structure. The total cost was given as £101,428, which seemed excessive for the length of the bridge involved; but in the absence of detailed statements, it was impossible to determine where the excess, if any, arose. Mr. Macdonald.

Mr. T. SEYRIG said, that in the erection of the Sukkur bridge, it became necessary to employ successively several different methods of work. This appeared to go a long way towards deciding whether the original design of the structure was entirely adequate. It was at once elaborate and complicated. The apparent simplicity of the general lines seemed in a large measure to have lost its advantages in the complication of the details. It had also led to the result that in the erection very heavy parts had to be handled. Except in special cases, it could not be advantageous to lift pieces weighing as much as 14 tons, and this would most probably have been avoided if the work on the spot had been more considered while designing. Bearing in mind the questions of ease and safety, and more especially economy of erection, the best practice would limit the weight of parts to be lifted to 2 or 3 tons, and this was particularly the case when rope tackle had to take the place of staging. As it was, the erection of a single span had necessitated not only some important wood staging, and a complete set of rope tackle, but also a temporary iron staging for the central span. The result told upon the total cost, which amounted, including special plant, to about Rs.811,000, or £57,450 for 3,316 tons. Making allowance for all special circumstances, distance, &c., £17 6*s.* per ton was certainly a very heavy price when compared with what had been realized elsewhere, owing to better provision in design. Mr. Seyrig.

The difficulties encountered (and so frankly stated by the Author of the Paper on the Chittravati bridge) during the sinking of the cylinders, were typical of such undertakings, and they seemed to raise once more the question whether it was really best to sink cylinder-foundations by dredging. It was true that in soft ground and at moderate depths it could be easily and safely managed; but the slightest accident would sometimes upset all provisions, often more than doubling both time and cost. In the Chittravati

Mr. Seyrig. foundations, the pneumatic process had been resorted to when emergencies occurred. It was barely satisfactory, the plant being old, and the cylinders often cracked through previous blasting operations. Under such circumstances, it was impossible to consider the work done, and its cost (443 rupees per lineal foot), as at all representative of what it should have been if the greater part of the sinking operations had been done pneumatically. The mean cost of sinking, deducting the portions done by hand-labour, was 42 rupees, or £2 19*s.* 8*d.* This price could certainly have been considerably lowered if pneumatic appliances had been used throughout, and it was moreover certain that the possibility of examining the ground during the process of cleaning the bearing surface, and of laying and ramming the concrete when the excavation was completed, were advantages which increased the value of the whole work to such an extent that Continental engineers now almost universally avoided any method which did not insure the examination of the foundation ground *in sitû*, and at the same time prevent the inconvenience and danger of depositing concrete under water.

Mr. Wilson. Mr. JOSEPH M. WILSON remarked that the two bridges under discussion presented widely different conditions in reference to design and facilities for erection. While the total length of the Chittravati bridge was considerable, the spans were comparatively short, and allowed the adoption of an economical type of superstructure, which called for no special comment, except that in noticing the American form of outline with its familiar name of "Murphy-Whipple," he could not but observe the absence of pin connections, which to an American mind would have considerably facilitated the erection. The engineer was to be commended, however, for the skill with which he had availed himself of the natural advantages of the location, in constructing the sub-structure as well as the superstructure, and for having successfully completed the work in what Mr. Wilson believed to be a very short time as compared with Indian work generally. It was well known that the development of pin-connected trusses of this type, having vertical compression and inclined tension web-members, had reached high perfection in the United States. The uncertainty of the strain, however, in the inclined web-members towards the centre of the span, where ties and counters occurred in the same panels, together with other considerations, had led some engineers, himself among the number, to favour the adoption of triangular trusses, where certain web-members were exposed to alternating stresses of tension and compression, according to the position

Mr. Wilson. of the moving load. He had had considerable experience in the inspection of bridges in service, and had never observed any wearing action in the pins of structures of the "Murphy-Whipple" type, even after years of use, and in cases where the pins and links were not designed according to the most modern ideas in reference to areas of bearing surfaces, bending moments, &c. His attention had been called, however, to the case of a bridge of his own design on the triangular system, where an action was noticed on the pin which had never been observed before. It was a small structure in which the effect of the variable live-load was severe as compared with the dead-load. After it had been in service for about five years a change in the alignment of the road necessitated the removal of this bridge to another location. In taking it down it was discovered that the pins had been worn into grooves at the bearings of the links, in some places as much as one-eighth of an inch in depth, these grooves being almost as clearly defined as if cut by a tool. The pins and links had been well proportioned for bearing surface, and it was evident that the result had been due to the turning of the pins in place. It was thought that the action of the alternating stresses in the links, first a push and then a pull, caused this rotary motion, and that if the pins had been secured from turning the difficulty would not have occurred. Where the stresses in each member of the truss were always of one kind in the same member as in the "Murphy-Whipple" type, giving a constant bearing on the pin, there did not appear to be this tendency to revolve.

He observed that the Sukkur bridge, in common with other cantilevers, presented an obvious mode of procedure in the erection, but the large sizes and weights of the members to be handled required careful treatment, and the work seemed to have been well carried out. As there were evident delays in the receipt of material from England, no criticism could be made on the time taken for the erection.

Mr. Hayter. Mr. HARRISON HAYTER, Vice-President, would, in the absence of Mr. Stoney, reply to the correspondence in so far as it related to the Chittravati bridge, and had not been noticed in his previous remarks. Much of the correspondence was from America, and it was useful to know the views of American engineers on English practice. In reply to Mr. Bouscaren, he assumed that Mr. Stoney had not used an inside curbing of masonry to assist in sinking the cylinders of the piers because he would not desire to contract the working space, and also, probably, Mr. Hayter. from the difficulty of fitting masonry to a surface where there were flanges, ribs, and bolts; this objection would not hold in the case of concrete which was used for the filling inside the cylinders, and which no doubt also would cost much less than masonry. As regarded the adoption of the pneumatic process for sinking cylinders, he had used it extensively, but he was opposed to it if it could be done without, especially if the cylinders were in deep water. Men working under air-pressure did so at a disadvantage both as regards progress and bodily discomfort, and often at the sacrifice of health. He did not believe that the cost would in any way have been lessened, as Mr. Seyrig also seemed to think, if the pneumatic process had been adopted throughout, and he considered Mr. Stoney had done well in limiting its use. Mr. Hayter had used elsewhere cylinders entirely of wrought-iron up to the conical length, but they added to the cost and were not so readily put together as cast-iron cylinders made in segments bolted to one another. All that was necessary was to make the bottom length of wrought-iron as he had done in the Chittravati bridge, and if this were made strong enough to take the strain, there was little fear, under ordinary conditions, of the cast-iron cylinders cracking during the process of sinking. Mr. Jules Gaudard was right in the supposition that Mr. Hayter had designed the bridge with independent girders throughout, instead of continuous girders, in order to facilitate erection. Continuous girders could not have been so readily dealt with, and would have involved more riveting, and of a more difficult character. Continuity also added to the complication where the sides of the girders were composed of struts and ties and not of solid platework, and there were more parts not duplicated. He did not believe that any saving in cost would be effected by connecting the girders together over two or more spans in a case like the Chittravati bridge. He preferred two cylinders instead of one for the piers. A better base to the piers was thereby secured, and the surface-friction was reduced to a minimum in sinking. In designing bridges for India, and in like places where the climate was hot, and where the locality of the structures was at a distance from manufacturing centres, the greater the facilities that could be afforded to the engineers who erected the work, the more probable it was that success would be ensured and expense saved. He was not aware that any injury had resulted during transit by the plates and angle-irons where they were separated for shipment being broken with splices and not square across. He had followed both plans, but he found that if the ends were well protected with temporary timbers they reached their destination uninjured, and sounder work (as Mr. Gillott remarked) was the result when the girder was erected. He believed that there was no more likelihood that there would be loose rivets with native riveters than there would be if Englishmen were employed. There was no reason, however, why machine-riveting, either actuated by steam, water or air, should not be introduced in India as well as in America or England. He agreed with Mr. Hogg that the surface-friction encountered in sinking cylinders depended not only upon the nature of the strata penetrated, but also upon the cylinder being kept vertical, and the table Mr. Hogg had sent was instructive. Compared with that given by Mr. Stoney it appeared that the surface resistance was much less in the case of the Chittravati bridge than in that of the Dalmarnock bridge, owing no doubt to the different conditions. Mr. Macdonald, from his connection with America, was probably unaware of the very proper restrictions imposed upon engineers with regard to iron bridges in England and India. The Chittravati bridge was designed with the authorized factors of safety, and with a limited allowance for deterioration, which was desirable now that iron bridges were being continually renewed or strengthened owing to oxidation and decay. He would not, as he had already said, sacrifice efficiency by making girders of the excessive depths sometimes introduced in America, but which would not be tolerated in England or India, and he considered that no material saving in weight would result if the deep girders were so braced and tied as to be as efficient as they could be made. The equation between the cost of the piers and superstructure was such, he believed, as to give the best results for the completed structure. Although the length of the spans was the result of a suggestion from India, they quite met his approval as being the most economical to adopt considering the conditions; and that the design of the Chittravati bridge was as suitable as could be devised seemed to be evident from the fact that whilst it was a rigid structure the cost was very low compared with that of other bridges across rivers of a like kind. Mr. Wilson seemed to prefer pin-connections, which Mr. Hayter had largely used, and which Mr. Wilson said would to an American mind have facilitated erection. The practice, however, in England differed from the American in this respect. The use of pin-connections was now rather the exception here than the rule, and the reasons for this preference should apply with greater force to iron structures which had to be transported

Mr. Hayter.

Mr. Hayter. to and erected in India. The accurate fitting necessary and the liability to alteration of shape by transport and climatic changes were unfavourable to the adoption of pin-connections on a large scale in such places as India.

16 December, 1890.

Sir JOHN COODE, K.C.M.G., President, in the Chair.

The discussion upon the Papers by Mr. Robertson and Mr. Edward Stoney, descriptive of the Sukkur and of the Chittravati Bridges respectively, occupied the evening.