Paper No. 6320

Development in overhead electrification of railways as it affects the civil engineer †

by

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Discussion

Mr A. K. Terris (Chief Civil Engineer, British Railways, Eastern Region) said that the Paper covered almost all the problems connected with electrification in its purely civil engineering aspect but by no means all the work embraced in modernization of the railway. For example, when large works such as bridge renewals were being carried out, the track possessions required for such work should be made use of as much as possible for carrying out all the other necessary works on the railway, such as normal and accelerated maintenance and improvement works in advance of electrification and also work carried out by the other technical departments. Very detailed planning and staging was, therefore, necessary to ensure that such projects could be carried out at the appointed times and within the time allowed so as to curtail delays to traffic to the absolute minimum.

101. The Author had, in particular, shown that perhaps the main problem of electrification was concerned with clearances, particularly at overbridges. He had referred to the consequences of the pantograph design and to the various devices and methods which had been engaged upon to avoid as much as possible the consequences of altering the statutory level of roadways over bridges and thus to design overbridges with a very small depth of construction. Out of this task had developed the partially prestressed bridge deck described so fully in the Paper.

102. Mr Terris referred to the size of the pantograph and compared the problem in the United Kingdom with that which obtained on the Continent where, of course, the construction gauge was much more generous. The clearance diagram shown in the Paper indicated how much better the clearance problems would become if the pantograph could be shortened or insulated at the horns. Alternatively, if a reduction of the trace of the wire could be seriously considered, augmenting the pantograph rubbing face with additional strips to take care of the additional wear, then this would appear to be a solution.

103. Clearances were, of course, particularly restrictive in the region of quarter-points in arch bridges. Of a total number of 2,400 overbridge spans on the Eastern Region, about 50% were brick arches with sometimes several spans traversing tracks to be electrified. This indicated the reason why so many methods of dealing with the clearance problem at bridges had been tried as described by the Author. Every case was, of course, considered on its merits.

104. A little publicized aspect of electrification, which was of paramount importance to progress, concerned works carried out by the British Transport Commission for highway authorities and other bodies ahead of, or concurrently, with electrification.

105. The Ministry of Transport and Civil Aviation Circular No. 748, "Railway electrification of bridges", issued in 1957 to all highway authorities, indicated that increased headroom provision would be necessary under many of the bridges over the lines to be electrified, and stated that this offered opportunity to the authorities for concurrent highway improvements. The Minister had recently stressed that he was prepared to consider now, for approval, any schemes which the highway authority wished to sponsor. This opportunity was not being ignored by the local authorities and, in practice, the magnitude of this work had reached such proportions that it could

† Proc. Instn civ. Engrs, vol. 12, p. 125 (Feb. 1959).

be said to have a very considerable effect on the programme of the Commission's work, and it was very difficult to merge the interests of both parties. On the Eastern Region alone the value of bridge works being carried out for highway authorities in the current year was in the region of £1 million. The procedure for financing individual public works of this kind was not readily adaptable to rapid development such as the British Railway Modernization Plan called for, involving a number of local negotiations, often of a protracted nature, and all were regularized by individual agreements. In these negotiations it was necessary to consider existing Acts referred to by the Author, land matters, future maintenance and renewals, etc., and the negotiations generally tended to get very involved and extenuated.

106. As the highway programme gained momentum, it was therefore increasingly evident that a simplification of the procedure now in being was most desirable to ensure efficient working at a time when the financial interests of both parties were of a national character, and the necessity for the stringent negotiations of the days of private ownership, on the part of the railways, did not prevail to the same extent.

107. On the subject of tensile stress in concrete, Mr Terris had at first, like his predecessor in office, been frankly sceptical, but he had been convinced that the practice was sound after witnessing a series of punishing tests on beams in a deliberately cracked condition, and his belief had been reinforced by subsequent tests to failure which had been carried out at Liège (see Table 3) which, in all cases, had so nearly approached the calculated ultimate resistance of the beams concerned without taking into account any fatigue.

Brigadier C. A. Langley (Chief Inspecting Officer, Railways, Ministry of Transport and Civil Aviation) noted that in § 2 the Author had referred to S. R. & O. No. 827. This was the Railway (Standardization of Electrification) Order 1932, and had been promulgated to give statutory authority to the recommendations of the Pringle Committee on electrification. There was no mention in that Order of A.C. electrification; consequently, when the Minister had given his approval to the B.T.C. proposals for 50-c/sec A.C. electrification that Order had had to be revoked. It had not been replaced, because the Inspecting Officers relied on the powers under the 1933 Road and Rail Traffic Act, by which the Minister could give approval to electrified lines and could authorize the inspection of the new works before they were brought into use. The Minister's Requirements laid down the standards which they expected to find on such inspections, and the 1950 edition covered D.C. electrification but not A.C. The Inspecting Officers had, however, refrained deliberately from publishing an official amendment to those Requirements, because they wanted to have some experience of what was, for Britain, a new form of electric traction before committing themselves too deeply.

109. For example, in § 27 of the Paper, reference was made to the use of special insulation at bridge soffits at places where there were unusual difficulties. Unfortunately, up to the present that insulation had not been perfected and it had therefore not been brought to the Ministry for approval. It was to be hoped that the electrical engineers would soon solve that problem, because the Inspectorate appreciated very fully the difficulties facing the civil engineer in coping with the clearances needed for A.C. electrification.

110. The Author had referred to the high standard of permanent-way maintenance needed before a line was electrified. Those standards had always to be maintained, because experience had shown the damaging effect which was produced by the nose-suspended electric motor with its unsprung load.

111. Brigadier Langley noted the statement in § 50 that "It is intended to work up to 250 lb/sq. in. tension in railway underbridges . . . can be severe". Such a proposal had not yet been made to the Ministry so he would not comment except to say that the Ministry were always ready to encourage economy of construction provided that it did not affect safety.

112. In § 79 there was a reference to 6-ft-high solid parapets on overbridges. He

appreciated that they might be unsightly in places, but it was necessary to protect the public from themselves. A mischievious person could be a great menace not only to himself but to others, so that an endeavour should be made to put temptation out of his way. The Ministry were, however, open-minded about this, and already they were considering modifications to the height of parapets on new motorways, from which pedestrians and cyclists would be specifically excluded by law.

113. In § 86 reference was made to the reassuring results obtained in testing prestressed concrete masts under collision conditions. Brigadier Langley recollected a test made on a prestressed concrete mast which resulted in complete failure of the mast to withstand a collision. On the other hand, collision with an overhead structure of massive design could cause severe damage to a railway train. What was needed was a mast designed to take the stresses imposed by overhead equipment and also to withstand a reasonable shock, but not a heavy collision.

Mr J. S. Campbell (Assistant Civil Engineer, Works, British Transport Commission) referred to § 9 of the Paper, in which it was stated that "The standing of British Railways with regard to other authorities and the public at large is governed by Acts of Parliament". The Author had gone on to quote from Section 86 of the Railway Clauses Act of 1845 that:

> "It shall be lawful for the company to use and employ locomotive engines or other moving power and carriages and wagons to be propelled thereby . . ."

and had hopefully added "which would appear to give Railway Managements all the authority they now require." Mr Campbell hoped that the Author was right.

115. In § 14 the Author referred to P/d values which helped the civil engineer to judge if the ratio was too severe. It was based on the Hertzian formula. Mr Campbell was not so pessimistic as the Author on this matter and felt that a great deal of work had still to be done on it. He wondered if some of the Author's conclusions might be questioned. For many years civil engineers had been complaining to their mechanical colleagues about the severe affects of the steam locomotive on track; the civil engineer had to provide and maintain the track for it; bridges, too, were the civil engineer's responsibility and with more than 35,000 underbridges it was no mean task. A new form of propulsion had now been introduced, Diesel or electric, and, somehow or other, the effect was found to be more onerous than that of the steam locomotive. It might be that the actual shape of the new locomotive was partly responsible. Instead of the normal more resilient form of the steam locomotive with its tender, one had a long four-sided girder with four stiff sides, like compartment stock but much stiffer on all the four sides, with a bogie at each end, resulting in the effect of two locomotives rather than one passing over a given point. In considering the effect of the steam locomotive, with the downward weight of its compact axle-loads, as with the 4–6–2 (Pacific) type, there was no change in sign in the effect of the load on the track, whereas with the 60-ft-long Diesel or electric locomotive with two axles at the front and two at the rear (or three and three) there was the possibility of the formation of a wave condition. Mr Campbell suggested that the biting of the driving wheels into the upward gradient of the wave might have a greater effect on the rail than the P/d ratio. Research workers were looking into that factor and it was hoped that they would be able to provide more information about it in the near future.

116. From the information available at the moment, in a report not yet officially published, it was found that vehicles of the type which he had described—60 ft long with bogies at each end—even taking the old-fashioned dining cars, with axle weights of 17–18 tons, the effect of those vehicles on short rail-bearing members of a bridge was as onerous as in the case of a 4–6–2 steam locomotive. A number of peculiar effects, arising from the wheel arrangement of vehicles, were being discovered, and it might be wise for civil engineers to wait a little before making too serious a pronouncement on what the maximum P/d ratio should be.

117. Reference had been made to the prestressed concrete decks. The Bridge Sub-Committee of the British Transport Commission (of which Mr Campbell was Chairman) had considered the designs carried out by the Eastern Region. While they would not regard the "wafer" deck, referred to in § 54, as a job for everyday use, it had its very definite function. The cost of the wafer deck would usually put it out of competition, but where the construction depth was limited and nothing else could be done it was a sound job. The wafer deck had been considered in all its aspects by the Bridge Sub-Committee, and that had involved the partial prestress which had been mentioned. They were satisfied with the soundness of the design in that respect and the appropriateness of its use both normally and for the wafer deck. That conclusion had been approved by the Civil Engineering Committee of the British Transport Commission and finally by the Commission itself, so that he could speak for the Commission in saying that this kind of design was accepted as a useful and safe method of resisting loads.

118. Reference had been made by Brigadier Langley to the 250 lb/sq. in. tension in railway underbridges. The Author, in his usual careful way, had said in § 50 that "It is intended to work up to 250 lb/sq. in. tension." What he should have said was that when an engineer designed members for a job he could never, by trial and error, get the exact answer right away, and so had to set down some limits as between no tension and something else. The limits between no tension and 250 lb/sq. in. represented what would be practised in normal design, and Mr Campbell suggested to Brigadier Langley, who had referred to the fact that nothing had been put up to the Ministry on this matter, that it had not been thought necessary to do so since it was a design factor and was within the working range of no stress up to 250 lb/sq. in.

119. Brigadier Langley had also referred to the 6-ft parapet. The Commission had been placed in an awkward predicament by having to put a 6-ft parapet on bridges over the track. They had been in negotiation with the Fine Art Commission on one or two aspects of it and had found them very willing to help. Their advice was that the 6-ft parapets should be made to look attractive—which was precisely what the Commission had been trying to do for years.

120. Mr Campbell thought it might be more productive and perhaps, in the long run, cheaper, to conduct a national campaign in the schools to educate young people in the danger of interfering with the overhead wires. Unfortunately, however, some youngsters, whatever barrier was placed in their way, would surmount it. Mr Campbell hoped that the Ministry would help by extending their waiver for motorways, where pedestrians and cyclists were not permitted, to the normal overbridge, with perhaps some reservations for bridges near schools.

Mr R. L. McIlmoyle (Assistant Civil Engineer (Modernization), British Railways, London Midland Region) referred to the information given in § 28 on the cost of clearing overbridges. On the London Midland Region the price range for overbridge clearance had been from £2,400 to a maximum of £80,000 for a large multi-span bridge. The cost per mile quoted by the Author, £15,000, might seem a staggering sum, but on the Liverpool–Crewe line of the London Midland Region, where the price range was from £4,350 to £60,000, the cost per mile was £19,500. For the whole Euston-Manchester-Liverpool scheme the cost per mile was expected to be rather lower, £14,600, but that was still a substantial figure. At the northern end, between Weaver Junction and Carlisle, the cost per mile might be expected to be substantially lower, but it was still of the order of £14,000.

121a. On the question of statistics, Table 1 was very helpful and Mr McIlmoyle gave similar figures for the London Midland Region in Table 7. The comparison with the Continental figures was staggering and explained the figures he had quoted earlier of £14,000 to nearly £20,000 per mile.

122. Mr McIlmoyle agreed with the Author that the lowering of tracks should be avoided except as a last resort. The London Midland Region had had some experience of it, and it had taken a great deal of time and money.

123. The reconstruction of arch bridges was referred to in § 61. Between Crewe and

Manchester there were twenty-eight three-span arch bridges, and on eight of these reconstruction of the side spans as well as of the centre span had been necessary. Efforts had been made to devise a standard design which could be used either for the centre span or for three spans. In one design the side arches had been tied back while dropping and replacing the centre span. An alternative design had been developed in which the new arch was cast on top of the old and the old arch then removed by explosives. This method had also been used for the reconstruction of ten single-span arches.

TABLE 7

	Bridges per route mile	Density in terms of (*a*) in Table 1
Manchester to Crewe	1·89	5·56
Liverpool to Crewe	1·65	4·85
Euston-Manchester-Liverpool	1·44	4·24
Weaver Junction to Gretna	1·37	4·00

124. With regard to the reconstruction of bridge decks, on the London Midland Region it had usually been found that arch abutments were too weak to raise them economically and put in flat decks, so the arches had been replaced by new ones designed to give clearance and retain stability of structure. Where flat decks had been possible they had not employed partial prestressing of the type described in the Paper, but a prestressed design had been employed. Those decks cost about £16·8 per sq. yd. The wafer design shown in the Paper was ingenious, but such shallow construction decks had not been necessary on the London Midland Region.

125. The Author had given a good deal of detailed information about the dropping of arches. The London Midland Region had used three methods. The first was similar to one of the Author's methods. Windows had been cut at haunches and a steel ball, dropping in the middle of the arch, demolished the remainder. This was a quick method. The second method had been to use pneumatic drills to cut the old arch from under the new arch, but this proved slow and costly and had been abandoned in favour of the third method. The third method was to demolish the old arch from underneath the new arch by controlled explosives. This was found to be quick, safe, and economical. The arch was completely demolished in 8 hours and one track cleared in 3 hours. Two ounces of gelignite was placed in horizontal holes 3 to 5 ft deep. Charges were fired at ½-sec. intervals with an average of six charges per shot.

126. They had so far dealt with only one tunnel, a cut-and-cover job with shallow cover. It had been opened out and replaced by two bridges, one of which carried a large roundabout and required a bridge 170 ft wide between parapets. This had been the quickest and most economical way of doing the job and it had been possible to increase the lateral clearances which were sub-standard.

Mr J. R. Dallmeyer (Assistant Civil Engineer, British Railways, Eastern Region) gave some details of the electrification schemes of the Eastern Region in the area to the north-east of London (Fig. 22). The line from Liverpool Street to Shenfield had been electrified almost 10 years ago, and the extensions from Shenfield to Chelmsford and Southend were completed in 1956, pressure being at 1,500 volts D.C. In 1959 they would be converted to 50-c/sec A.C. high voltage. The line from Liverpool Street to Bishop's Stortford, with its branches, was being electrified on the 50-c/sec A.C. high-voltage system and would be completed in 1960. The line from Fenchurch Street to Shoeburyness and branches was being electrified at the same cycle and voltage and would be completed in 1961. In addition to the lines shown in Fig. 22, the line from

Colchester to Clacton and Walton was being electrified, again on the high-voltage 50-c/sec A.C. system, and was to be opened as an electrified line for public passenger service on 16 March of 1959. It would be the first public service at 25 kV, 50 c/sec to be run in the British Isles.

128. The line from Manchester to Sheffield and the branch to Wath had been electrified as a 1,500V D.C. system between 1952 and 1955. These schemes represent a total single-track milage of 550.

129. The P/d ratio (§ 14) concerned the intensity of pressure between two cylindrical elastic bodies which induced shear stresses in them. In 1947 consideration had been given to altering the head radius of the rail from 12 in. to 9 in., and in those deliberations

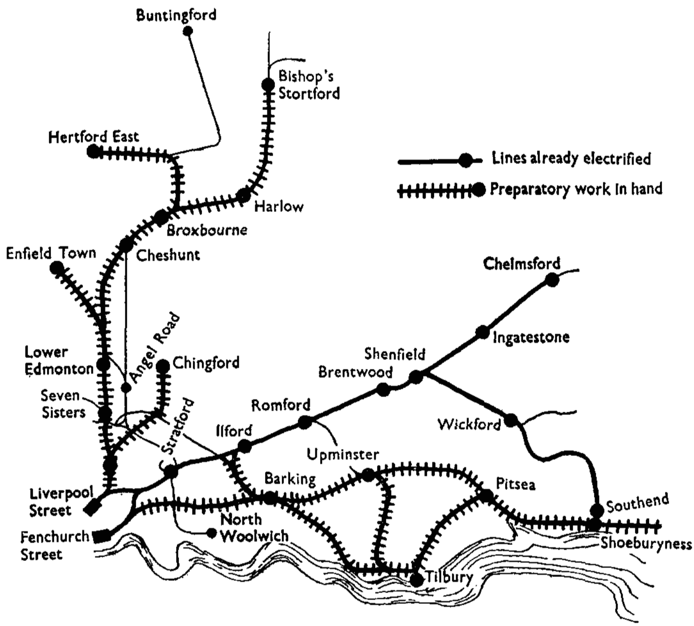

FIG. 22.—NORTH-EAST LONDON ELECTRIFICATION ON BRITISH RAILWAYS, EASTERN REGION

there had come to notice a report from America on Hertzian stresses, made in 1930, which suggested that when the shear stress in the material exceeded half the ultimate tensile stress of that material damage would be caused to the under-surface of the material due to shear fatigue when it had undergone something like 2,000,000 applications. When that theory was applied to rails, assuming 9-in.-rad. head, and using the loads and diameters of wheels as explained in the Paper, damage was expected to occur only when P/d reached or exceeded a figure of 4·5. There had not yet been, so far as Mr Dallmeyer knew, any case of that description in Britain. In the past, of course, the P/d figure had always been below 4·5. As Mr Campbell had pointed out, the steam locomotive had never given any trouble in this respect. That was not because its wheels were bunched together but because its P/d figure was quite small. If the P/d figure and the 2,000,000 applications were applied to the Manchester-Sheffield and Wath line the P/d was 5·4, which was high, but the intensity of traffic on this line was not sufficient to give 2,000,000 applications in less than about 23 years, by which time there was no doubt that the rails would have been renewed.

130. Referring to a line with more intensive traffic, that from Liverpool Street to Shenfield, the *P/d* figure was in fact 3·7—a little less than that given by the Author—and, if the theory referred to was correct, the stresses which would cause damage were not in fact reached. However, the intensity of traffic was such that the figure of 2,000,000 applications would be reached in about 2½ years.

131. Instances of the rails being not truly horizontal would, of course, increase the *P/d* figure at the point where the wheel was lifted out of a depression, but it would be expected that a depression in the rail would not exist for long enough—i.e. 2½ years—for the Hertzian stresses to have an effect.

132. The Liverpool Street–Shenfield multiple stock, which had not a very heavy *P/d*, had some of the effects thought to be attributable to high *P/d* ratio. There was, for example, deformation in the form of short corrugations, with a certain amount of shelling on the gauge side. Those rails had, however, been down for 6 years, and there was only 4 to 6 in. of slag ballast between the underneath side of the sleeper and the concrete deck of a viaduct. The trouble had necessitated the removal of the rails. In another case (again on 6-year-old rails) a long flat on the rail head was followed by an adjacent stretch of slight undulations. Both conditions, Mr Dallmeyer believed were caused by the type of rolling stock running over the line, with its heavy unsprung load combined with slipping of the wheels which occurred in places where acceleration and deceleration took place, and with the lack of sufficient ballast underneath the sleepers.

133. Those two cases were being investigated by the Railway Research Service, for they were much more severe than anything of the same nature which had been experienced with steam traction.

134. In all recent electrification projects it had been the aim to place 12 in. of good clean ballast beneath the sleepers, and with elastic fastenings, rubber pads, and long welded rails it was hoped to achieve a satisfactory track. This type of track had been installed in considerable lengths and under steam and electric multiple-unit stock the condition so far was excellent.

Mr Ronald Bridgman (Senior Engineer, Sir William Halcrow & Partners, Consulting Engineers) asked if the Author would place any upper limit to the span for which he would use the design referred to in § 45. Normally, one would expect that the weight of solid decks over 40 ft would be excessive and some form of hollow deck would be a cheaper form of construction.

135. In connexion with the comparison of the costs of the different designs which the Author quoted, would he give some further information on the cost, the span, and the loading of the three designs mentioned in §§ 41, 51, and 53? The span appeared to be mentioned only in the case of the § 51 design; the loading was not mentioned for any of the designs. Had the approval of the section of the Ministry of Transport responsible for road bridges been given for tension in the concrete.

136. Could the Author equate the costs for the three different designs to one particular year in order to eliminate the effect of the general rise in prices between 1949 and 1956. A true comparison of the cost of different designs was important in view of the large expenditure involved in raising bridges for the electrification programme.

Mr Edgar Claxton (Assistant Electrical Engineer (Development), British Transport Commission), referring to previous comments on the *P/d* ratio, said that the new 50-c/sec 25-kV A.C. electric locomotives, which were being built for the British Transport Commission at the moment were four-axle locomotives and were all fitted with fully spring-borne motors with a view to easing the conditions on the rails. Whether or not fully spring-borne motors did in fact achieve any great improvement was a matter still under discussion and research, but the design was the best that could be devised and it might make some improvement.

138. Cable routes were mentioned in § 22. Where it was possible (which was not very often) cables were buried, but where that could not be done they were laid in

surface concrete troughing or in split asbestos tubes on posts. Undesirable proximity to the cess rail was kept very much in mind, and when direct burial was possible they were kept well away from the track.

139. Mr Terris had referred to pantographs and stagger of the contact wire and the Author had dealt (§§ 25, 26) with the saving which he alleged could be made by having insulated horns.

140. The reasons why British Railways could not use the insulated horns, starting in the same position as on the French system, were tied up with the questions of stagger and curvature. In the first place, British Railways included much very complicated trackwork, with corresponding complication overhead, and their space was very much restricted. The French and all the other Continental railways had by comparison all the space they wanted and so could simplify their overhead equipment. Secondly, a standard maximum span of 240 ft had been adopted in Britain on the score of economy and reduction in the number of structures, whereas the French worked on about 60 m (less than 200 ft). The longer span affected the situation under wind conditions, but the British Transport Commission worked to a normal maximum stagger of ± 9 in. on tangent track, and on curves to a normal maximum stagger of 15 in. On point-and-crossing work they went to an absolute maximum of 17 in. All those figures had a tolerance of probably $\pm$ 2 in. The French, by having easier point-and-crossing lay-outs, could restrict their staggers or the points at which they had to collect current compared with what could be done in Britain.

141. The stagger on tangent track did not affect the issue. There was ± 9 in. available to spread the wear, but it was not essential, and could be varied when desirable. It was the curves and point-and-crossing work which were the controlling factor. Closing-in the structures on curves enough to keep the stagger down to the sort of level in question would bring them very close together and would still leave point-and-crossing work outstanding. The use of an insulated horn had been considered but, to avoid burning, it could only start outside the point at which current would be collected. Compared with a length of 13½ in. on the French pantograph pan there would be only 8½ in. of insulated length and the electrical stress gradient over that length of glass fibre (often covered with water and copper and carbon dust) with a 3-in. air gap would give an electrical engineer cause for considerable alarm. The normal distance it was expected to require to maintain insulation with glass fibre at 25 kV was 80 in. without an air gap. The conclusion had been reached, therefore, that it would be dangerous, because of stagger on curves and point-and-crossing work, to start an insulated horn where the French did, and if started where it *could* be started it would be of no use in helping the clearance problem. The French had used insulated horns on certain lines, but Mr Claxton believed that they had experienced burning at the point of attachment such that the insulated horn broke off.

142. On behalf of the electrical engineers of the British Transport Commission, Mr Claxton said that they were very conscious of the need to save the cost of raising bridges and the like and the cost of clearances, for they wanted to carry out electrification as cheaply as they could, because they would then have more electrification to do. As a civil engineer he had every sympathy with the civil engineering side and with the problems which civil engineers had to meet in getting their bridge decks between the Scylla of the overhead equipment and the Charybdis of the highway authority.

Mr T. H. Rosbotham (District Engineer, British Insulated Callender's Construction Co. Ltd) said that some of the major works which the Author had done on the Eastern Region had been carried out not only for the electrification proper but also to speed up the efficient working of the lines in question. When such work could be done well in advance, it eased the job of the electrical engineer. For instance, the Author was pressing for a reduction in the side clearance of equipment. This, of course, was against the requirements of the electrical engineer, who wanted as much tolerance as possible, so that when tracks were altered there was still the possibility that the equipment would register satisfactorily over the tracks.

144. There were instances where, although overbridges had been raised to the maximum extent which would satisfy the requirements of the Ministry of Transport, there was still not sufficient clearance to provide the necessary electrical clearance over the equipment. Special consideration was now being given to the provision of additional insulation between the overbridge and the equipment. Other problems included the installation of foundations for the overhead structures. In that respect the ground alongside the line appeared to be made up of almost every type of material, and there was a particular problem on embankments, where the make-up of the ground could be virtually anything and special types of foundation had to be employed. In certain instances Mr Rosbotham believed that the civil engineer had incorporated a special process of grouting embankments so that they were more stable when the overhead structures were installed. That process, he believed, was very expensive and could not be resorted to in all cases, so that a special type of foundation, to take into account a certain amount of subsidence or movement of the embankment, had had to be designed.

145. The Author appeared to favour the introduction of the prestressed type of structure, which had been tried out on the Southend line in a very small way. There were difficulties in the introduction of this type of structure, in that it was heavy by comparison with the equivalent steel section and was heavier in cross-section and therefore more cumbersome to install; fewer could be carried to the site on each wagon and erection was more difficult. Provided, therefore, that steel was in reasonable supply, Mr Rosbotham suggested that steel structures should continue to be used.

Dr P. W. Abeles (Assistant to the Chief Civil Engineer, British Railways, Eastern Region), referring to the comparison in § 99 between fully and partially prestressed deck slabs, mentioned a similar comparison which he had made[3] as early as 1941 and which, he believed, was the first of its kind. Three I-beams of the same cross-section and the same high-tensile wire reinforcement had been compared. In the first beam all six wires had been tensioned; in the second beam only two had been tensioned; and in the third beam none had been tensioned. All beams, however, had had the same ultimate resistance. Gueritte had replied[4] that in the second beam "strong cracking existed" under working load corresponding to a concrete tensile stress of 771 lb/sq. in., and consequently this type of design had been considered to be inadequate and undesirable. Subsequently, however, extensive tests had proved that in prestressed beams with well-bonded pretensioned wires, cracks did not become visible before the concrete tensile stress became 1,000 lb/sq. in. and even then they were so fine as not to be noticeable except by a skilled eye.

147. The Chief Civil Engineer's department of the Eastern Region of British Railways had, for the past 10 years, called for performance tests, at which the test loading corresponded to a concrete tensile stress of 750 lb/sq. in. For example, the new electric stock shed at Ilford, had required eighty-eight I-section beams of spans of 60 to 66 ft designed for a working-load tensile stress of 750 lb/sq. in. Normally only one beam of each line in a pretensioning bed was subjected to an acceptance loading. At Ilford, however, only one beam had been cast in each bed, and consequently all the eighty-eight beams had been successfully tested.

148. Under test load, corresponding to a tensile stress of 750 lb/sq. in., visible cracks must not occur. During the past 10 years almost all the beams tested had complied with this condition. Where they had not, it had been established either that the initial tensioning stress had been too low or that shrinkage cracks must have occurred before application of the prestress.

149. Dr Abeles illustrated an Austrian spun concrete mast placed near Innsbruck on the Arlberg line, based on his own design. The mast was not prestressed and had been designed for ultimate load. High-strength steel (approximately 140,000 lb/sq. in.) had been used, and as a non-prestressed construction the development of fine hair-cracks could not be avoided. Nevertheless, it had stood up very well for the past 25 years.

150. A prestressed mast of the kind shown in Fig. 19 was capable of resisting a good measure of impact. Such a mast had been subjected to a glancing blow by a brake van of 20 tons weight, projected into the mast at a speed of approximately 10 m.p.h. Afterwards the mast had been capable of carrying the entire working load, in spite of the cracking caused by the impact. The previous speaker had stated that he would prefer steel, but Dr Abeles doubted if any engineer-in-charge of maintenance would prefer the steel mast which had to be painted. While it was advisable to avoid tension in diplomacy and at home, it was beneficial in prestressed concrete under maximum working load, so as to obtain some kind of flexibility. Deformability and rigidity could not be combined. If it were desired to have some resistance to shock, it was essential to have flexibility.

151. Dr Abeles was grateful to the late Mr J. I. Campbell, M.I.C.E., former Chief Civil Engineer, and the Author, for their early recognition of his ideas on partial prestressing and for securing its introduction more than 10 years ago. Having just returned from a visit to the United States, Dr Abeles thought it worth mentioning that this type of construction had been used to a great extent in that country for floor and roof construction during the past 3 years, for the purpose of reducing the troublesome camber, full prestressing being provided for dead load only.

152. The basic idea of partial prestressing agreed very well with the modern trends of design, based on a new philosophy which omitted consideration of permissible working-load stresses but required full serviceability under working load and a definite factor of safety against failure. This allowed the introduction ofa construction which was both safe and economical and to which Brigadier Langley should not object. To obtain serviceability with partial prestressing, it was obvious that the maximum possible losses of the prestress must be considered. However, in many cases too-low losses had been taken into account when full prestressing had been intended, so that tensile stresses must occur in spite of the effort to avoid them, and a construction might be obtained of a lesser degree of serviceability if the concrete flexural resistance was not available, i.e. if shrinkage cracks had occurred before prestressing.

Mr A. I. Emerson (Assistant to Chief Civil Engineer, British Railways, North Eastern Region) said the Author had emphasized the importance of providing permanent way of a high degree of excellence for electrification, and that might be amplified to cover the provision of a uniformly excellent standard of permanent way having no potentially weak places. The most common source of weakness was the slack which occurred at many overbridges and which was nearly always associated with a deficiency of ballast for a considerable distance on either side of the structure. On a main line electrification scheme such as that on the North Eastern Region, where, in contrast to the Author's work in urban areas, there was a considerable mileage in country which was not built-up, a reasonable quantity of extra work on the road approaches usually enabled a uniform track profile to be attained without any slack remaining at the bridge, together with a minimum of 12 in. of ballast underneath the sleepers throughout the profile; thus the final track level could often be the starting-point of an overbridge scheme.

154. Mr Emerson then showed a slide illustrating how, in the longitudinal profile of approximately a mile of track, the slacks could be related to the general ballast requirements and to other lineside features. It was necessary to ensure that the formation could accommodate track lifts at the new levels, allowing space for cable ducts, signal apparatus, and stanchions for overhead equipment. All these items should be in positions fixed beforehand with the department concerned or agreed as appropriate for various site conditions.

155. Further slides illustrated typical cross-sections with standard dimensions for earthworks, ballasting, and lineside equipment, which took into account requirements for on-track and off-track plant and which embodied types of equipment designed for rapid mechanized installation.

156. Standards of this nature having been agreed prior to electrification, firm estimates and programmes for their attainment could be dealt with systematically,

either geographically or in order of priority, depending on the time available. If time permitted it might be possible to associate the work with the Annual Programme.

157. The Author had mentioned several other permanent-way matters which needed development before electrification. Mr Emerson had enlarged on those which involved a great bulk of routine work and the early completion of which would lead to long continuous stretches being available for wiring. The work would also benefit Diesel traction in the interim period before electrification. The relatively few more difficult problems which arose in the built-up areas of a main-line scheme could be dealt with over the whole period which was available for civil engineering work and the wiring operation.

Dr K. Hajnal-Kónyi (Consulting Engineer) gave a short summary of the history of composite-construction design.

159. In 1943, Paul[5] had described the use of precast prestressed units in war-time bridge construction, and had concluded that the method had certain deficiencies. It was that type of design which had given Dr Hajnal-Kónyi the idea of an improvement, and in the discussion[6] on Paul's Paper he had put forward a suggestion which had now been incorporated in the Mr Sadler's Paper. In replying, Paul[7] had stated that that suggestion would "nullify the advantage" which he claimed for the type used by the Ministry of War Transport.

160. A long time had elapsed. Then the Eastern Region had taken up the idea, as would be seen from the Paper; but not so the Ministry of Transport.

161. In 1952, Thomas and Short[8] had described work carried out at the request of the Ministry of Transport. They had tested four types of bridge of composite construction: the slab-and-girder system without shear connectors; the slab-and-girder system with shear connectors; the jack-arch system; and a filler-joist system. All had been designed for the same load, but had given very different results so far as ultimate loads were concerned. It was obvious from the results that the slab and girder without shear connectors was very inferior, and so was the jack-arch system. The slab-and-girder with shear connectors was better, but by far the best was the filler-joist. Dr Hajnal-Kónyi[9] had pointed out the difference and suggested that, based on the research work which had been carried out, a solid slab would be the best. Thomas and Short, however, had maintained[10] that there were other considerations which governed the issue.

162. Dr Hajnal-Kónyi had mentioned[9] that he expected the same result to be obtained with solid-slab reinforced concrete or prestressed concrete composite construction. The filler-joist system was inferior mainly because of its high steel consumption and because it was never possible to get the concrete underneath the bottom flange of the joists compacted on the site, so that the danger of corrosion was very great, and the concrete would crack very badly if the steel got its design stress.

163. He had been surprised, therefore, when in 1955, Holland[11] had accepted the idea as a good solution, but with the qualification that "In order to provide adequate resistance in prestressed concrete slab decks to transverse bending moment . . . it will generally prove necessary to prestress the deck transversely", although Mr Holland had admitted that the arrangement of adequate grouting of the transverse cables might be difficult and costly. The transverse prestressing of such a bridge was a great nuisance, because it took a good deal of time and caused delay. On the basis of the previous research work by the Building Research Station, Dr Hajnal-Kónyi was convinced that such transverse prestressing was entirely unnecessary.

164. He had evidence of the efficient load distribution in the transverse direction from a bridge at Scunthorpe, which he believed was the first under-bridge in Britain to be built in this way and on which tests had been made.[12] The transverse reinforcement of this bridge was a small fraction of the amount now specified by the Ministry of Transport. It had been found that the maximum deflexion at the centre-line of the bridge was 0·02 in. and at the edge 0·019 in., so that the difference was almost negligible. This represented 1/27,000 of the span of the skew bridge of 27° and 1/12,000 of the span

at right-angles to the abutments. The result had convinced him of the soundness of the design without transverse prestressing. He had been pleased when Mr Holland had accepted the idea, but it was regrettable that the Ministry still insisted on certain details which put up the cost.

165. The last step in this development was a very important piece of research[13, 14] completed in 1958. Two bridge models, one without transverse reinforcement and the second with nominal transverse reinforcement, had been tested under the Ministry of Transport abnormal loading. The first had given a factor of safety of 2·7 and the second a factor of safety of 2·9 at failure. After those tests, he did not think that there was any justification for doubting the soundness of this type of construction without any transverse prestressing and with a substantially smaller quantity of transverse reinforcement than the Ministry specified. Dr Hajnal-Kónyi believed that a considerable sum of money would be saved on the road building programme if the Ministry would waive its extreme requirements and would take into account the results of the research which had been carried out.

Mr N. W. W. Cockcroft (Chief Engineer (Bridges), Sir Bruce White, Wolfe Barry & Partners, Consulting Engineers) observed that the Author had shown great ingenuity in producing very shallow structural slabs, but appeared to have pursued the objective of designing the shallowest possible slab regardless of whether or not it was the best solution of the particular problem. The design which used the least material was not necessarily the most economic, and it was not always possible to make savings in road works and other incidentals to balance the additional cost of going to the limit in reducing the depth/span ratio.

167. Figs 5 and 9 in the Paper showed cross-sections of two bridges, and in each case about one-third of the total construction depth appeared to have been wasted under the carriageway, where the heaviest loads occurred. By setting all beams at the same level and reducing the thickness of the surfacing, either the overall depth could have been reduced by between 6 and 12 in. or the useful depth of at least the carriageway part of the deck could have been increased by the same amount. If no advantage would have been gained by adopting the first of these alternatives—and in the case of the bridge at Stamford (Fig. 9) it would not have been possible, because of the depth required for pipe bays—would not some economy, or at least a reduction of vibration, have resulted from the second?

168. Vibration was undesirable in itself and might also give rise to fatigue, especially if resonance occurred. It was noted in § 60 and Table 4 that resonance had occurred in the dynamic loading tests of the experimental slabs and that the stresses had been considerably augmented thereby, even in the slabs with the relatively high depth/span ratio of 1/22. The natural frequency of the slabs was stated to be 375 c/min, which was uncomfortably close to the 6 rev/sec of an express locomotive, and the frequencies of some road traffic vibrations were of the same order. Had any observations been made on completed bridges to ascertain whether or not dangerous oscillations could occur under traffic? The tests carried out on the St James's Park Bridge[15] gave some indication of the effect which might be expected.

Mr J. H. G. Cook (Assistant, Chief Civil Engineers Dept, British Railways, London Midland Region), adding to what Mr McIlmoyle had said about the reconstruction of arch bridges in connexion with the electrification of the Crewe-Manchester line of the London Midland Region, submitted that when reconstructing an arch bridge there were advantages to be gained in making the new superstructure an arch also, since the substructure had been designed originally to take an arch and any modifications to it were minimized if the new structure so far as possible simulated the old. It had been with this in mind that the two methods of reconstructing arch bridges to which Mr McIlmoyle had referred had been evolved.

170. The first bridges to be dealt with had been the three-span arches with tracks through the centre span only, of which there were seventeen between Crewe and

Manchester. The method adopted had been to excavate trenches behind the abutments and drill holes in the abutments and piers through which had been threaded Lee-McCall rods, the rods being tensioned sufficiently to take the horizontal thrusts from the side spans, thus enabling the centre arch to be demolished without fear of the side arches collapsing. The centre arch had then been demolished by weakening it so far as possible with pneumatic tools. The centre of the arch had been reduced in section so far as could safely be done without fear of its collapsing prior to possession being obtained. When that had been done, the final demolition had been carried out by dropping a weight on to the arch.

171. In Table 5, item 3, the Author had described this method as "Noisy; significant vibration and somewhat uncertain in result." There was, it is true, some noise and vibration, but the actual operation lasted only some 3 minutes and as a rule only half-a-dozen or so blows from the weight were required.

172. The next to be dealt with had been the single-span arches, most of which had been faced in hard sandstone with large sandstone voussoirs. A method had been developed of dealing with these which required the minimum occupation of track—one occupation only, to demolish the arch—and which obviated the need for cranes, except so far as was necessary to deal with the skips of rubble. The method was to build the new arch in situ on the back of the old, and the old arch was demolished from underneath the new arch. Holes were drilled into the face of the arch—about eight holes, 3 ft deep and 1½ in. dia.; into each hole was placed a charge of gelignite of 1½–2 oz. The charges were detonated with fuses which delayed the detonation of succeeding charges by a little less than ½ sec. This had the effect of bringing down about a 4-ft strip of the old arch. The operation was then repeated.

173. In Table 5, item 1, blowing up, was described as "Suitable only in country areas." On the London Midland Region this method had been used in the centre of towns and in residential areas without any trouble. The Author had stated that 1½ lb. of polar ammon gelignite was required per cubic foot of sound brickwork demolished. That seemed excessive. Was such a big charge really necessary?

174. For three-span arches where more than one arch had to be reconstructed this method had again been successfully used. An added advantage in this case was that once the new superstructure had been completed each of the old arches could be demolished separately with a minimum disturbance to traffic, whereas normally where a three span arch has to be demolished it is necessary to block all lines to allow for the three arches dropping simultaneously.

Mr A. H. Jenkins (Assistant Engineer (Bridges) British Railways, North Eastern Region) said that on the North-Eastern Region of British Railways the bridge problems had been very similar to those on the Eastern Region, but there were differences in the way in which they had been met. The North-Eastern Region had started their programme with the requirement that a large number of small overbridges were wanted in a hurry, and with only a limited staff available for design and supervision. The construction depth had to be low, and a span/depth ratio of 30 or more seemed essential. The most trouble-free form of construction for the ordinary two-track overbridge was the joist-and-concrete type. Their technique, therefore, had been to demolish the arch (which was usually the previous form of construction) and build up the abutments from springing level (leaving a shelf for cables if necessary) and erect the joists; then to use simple shuttering suspended from the joists and premixed concrete brought on to the site, the surfacing being applied later as opportunity arose. Many bridges had been built on those lines, and, although theoretically uneconomic in steel, the ease with which they could be built with unskilled labour and the minimum of skilled supervision resulted in surprisingly cheap bridges with great reserves of strength. The average cost had been £20 per sq. yd of deck, but the main advantage was ease of construction and minimum of follow-up.

176. They had started by assuming that not-worse-than-existing-approach-road

gradients should be used, but, after discussing these matters with various road authorities, it had been found that these authorities were quite sympathetic towards the suggestion that sighting distance was much more important than road gradients, and they would accept a slight worsening of the gradient if given improved sighting instead. That had proved a valuable arrangement, and the railway engineers had found that they could often solve their difficulties by careful selection of profile. Their first step now was always to design such a profile and get it agreed by the road authorities before any other action was taken. In a typical case a gradient had been increased from 1 in 19·3 to 1 in 18·4 (visually undetectable) but the sighting distance had been improved from 183 ft 6 in. to 185 ft. This had resulted in a gain of 18 in. in road level.

177. Once the profile had been agreed, the next move was to take every advantage possible of the curvature provided. To do this a joist-and-concrete job was not so attractive, and the tendency was to use a prestressed concrete beam, particularly since suppliers had overcome their early manufacturing difficulties. On the North-Eastern Region prestressed concrete had been used from the start, especially for the small occupation type of bridge, which lent itself to the unit form of construction.

178. For occupation bridges of up to 75-ft span, the beams would be of rectangular cross-section and laid side by side. Advantage was taken of the road profile by deepening the beams toward the mid-section. They did not normally tension the concrete for public road bridges or use super-sulphated cement, because of the difficulties of close supervision with limited staff.

179. For the larger bridges of 80- to 120-ft span they had developed their own type of plate-girder span, used with steel cross-girders of "top hat" cross-section. On the brim of the "top hat" (the lower flange of the cross-girder) longitudinal prestressed concrete deck units of inverted-U shape were used. This gave a pocketed soffit with cross-girders at every 12 ft. Both main and cross-girders were cambered to take full advantage of the road surface and the side connexions were bolted with high-strength bolts.

180. The first stage in the construction of these bridges was to build four isolated concrete piers, each generally being founded on its own piles and capable of carrying all the reaction from the bearings. The steel main girders were supported on those piers. The demolition could be delayed until the last moment. The wing walls and abutments were independent of the piers and did not carry the deck load.

181. They had learned a number of lessons from this work. Simplicity of design was highly beneficial to trouble-free construction. There was usually little profit in designing a cheaper deck if it required special attention to the supervision of manufacture, loading, and erection. Agreements with the road authorities on profiles and road closure should be concluded at an early stage.

182. For demolition, explosives or cranes should be used rather than other methods. The design should take advantage of every inch of road camber. It was wise to bring into consultation certain public bodies, such as the National Farmers' Union, at a very early stage.

Mr J. E. Kelly (Managing Director, Wellerman Bros, Ltd, Sheffield) said that in 1949 his company had worked with the Eastern Region of British Railways in the construction of the first of the bridges to use the partially prestressed concrete superstructures and since that time had carried out a large number of similar reconstructions.

183a. In practically every case it had been necessary to maintain both road and rail traffic. Where the existing bridges were of sufficient width to accommodate double-line traffic, the work was carried out in two halves in the normal manner. In cases of narrow bridges or where road traffic was extremely dense, it was necessary to erect temporary bridges alongside but this was avoided wherever possible owing to the additional expense.

184. In the case of Mouselow Bridge on the Manchester to Sheffield line the carriageway had been only approximately 12 ft wide and had been in constant use by heavy traffic from the adjacent brick works. The difficulty of maintaining road traffic had

been overcome by erecting a temporary road bridge over the span to be reconstructed. The sequence of operations had been as followed, during one week-end. The deck had been stripped down to the extrados of the existing arch and a temporary bridge consisting of two lines of two 24-in. × 7½-in. R.S.Js placed at each side with 12-in. × 12-in. timber beams fixed to the underside of the R.S.Js and decked out in timber. On the Sunday night the existing arch had been dropped and cleared away so that on the Monday a temporary road could be opened to traffic and the work of reconstruction could proceed.

185. In the case of another bridge the existing construction comprised deep parapet beams supporting cross-girders. On this bridge which had been reconstructed in two parts it was decided to span the opening at the centre-line of the roadway and above road level, with 24-in. × 7½-in. R.S.Js and strap up the cross-girders at mid-span; it had then been possible to burn off the cross-girders in half-lengths for the reconstruction of one half of the bridge while traffic used the remaining portion of the roadway.

186. Turning to Bridge No. 1002 at Ipswich Road, Colchester, undertaken in two halves, there were two initial problems. In the first place, owing to the 34° angle of skew and fractures in the existing arch it had not been considered advisable to drop one half of the arch and allow traffic to use the remaining portion. Secondly, there had been the difficulty of placing the slabs in their final position owing to the angle of skew and the weight of the slabs, each slab weighing 40 tons, and they would have had to be handled approximately five times before being placed in their final position.

187. After consideration, it had been decided that it would be better to drop the arch as a whole after the bridge deck had been constructed at a higher level than its final position. The method had been first to excavate the spandrel filling for one half of the width of the bridge down to springing level, cut through the springing of the arch in isolated places, and build concrete piers on the existing abutments from springer level capable of carrying permanent sill beams and the superstructure. In order to obtain sufficient space between the existing arch and the new deck for the purpose of construction and also for dropping the arch, temporary hardwood packings had been placed on the concrete piers and the sill beams had then been constructed on these. The prestressed beams had then been placed by road cranes and temporarily supported through their length by strutting from the existing arch, while the concrete surround to form the wafer slab was poured. After the final stressing of the concrete the deck was ready to receive road traffic. A temporary tarmacadam surface had been laid on the concrete and traffic diverted over this half by forming temporary ramps at either end. The second half of the bridge had then been constructed in a similar manner. On completion of the deck at higher level the temporary struts had been removed from between the old arch and the soffit of the new deck and the whole arch comprising about 250 cu. yd had been dropped and cleared away on a Sunday possession. The sill beams and new superstructure, each half of which weighed approximately 150 tons, had then been jacked down to its final position, using four 100-ton jacks. The infilling of the abutments between the concrete piers had then been completed and the two halves of the deck joined together.

Mr Bernard Bramall (Scientific Officer, Research Department, British Railways) referred to the Author's statement in § 12:

> "The second main reason for improving the tracks is allied to the treatment received from electric locomotives, as they are generally designed at the present time".

189. Mr Bramall had been waiting for someone to say that the locomotives of which British Railways had the greatest experience, namely those on the Manchester–Sheffield–Wath line, were not to be regarded as typical of the present tendency in electric locomotive design. That had been stated at a meeting of the Institution of Electrical Engineers.[16] Those locomotives had been designed near the limit of adhesion, and showed how it was

possible to compress tremendous power into a relatively small machine, but unfortunately they suffered from the effects of wheel slip, which were exaggerated by the series connexions. He understood that the A.C. locomotives would not have series connexions. That was one way in which the D.C. locomotives had been hard on the track.

190. He felt that the remarks which had been made about the *P/d* ratio had not made it clear whether it was the effect on the head of the rail or the effect on structures under the track which was under consideration. With regard to structures under the track, he took his cue from Mr Campbell, who had said that locomotives with a particular type of wheel base had been capable of producing very high stresses in certain types of bridge. It was possible to throw some light on that. Recently a series of tests had been conducted, not particularly for this purpose, but it had been found that Diesel locomotives of the Co-Co type were producing high stresses in rail bearers. It could be explained most clearly by analogy to a flexible foundation slab on a yielding base under the action of two different types of loading: (*a*) a uniformly distributed loading, and (*b*) a point loading. Under (*a*), apart from possible end effects due to shear, there would be no bending induced in the foundation, but under (*b*) bending would occur and if the length of the slab was fairly large in comparison with the thickness there would also be an uplift on part of the slab.

191. Something of that kind happened on a bridge. Rail-bearing members could not be regarded as simply-supported independent joists. In some bridges they were obviously continuous, and elsewhere the cleating was such as to render them continuous. Whether or not it was desired to take continuity into account, it was there. The series of cross-girders acted effectively like a yielding supporting medium, and the diagram of bending moments induced in the rail bearer itself under a point load would be of a general form showing a point at the load position and in many cases an actual negative moment farther away from the loading corresponding with upward reactions on the cross-girders. Similarly, an influence line for moment at any point in that rail bearer could show a negative range.

192. That helped to explain the difference between the steam locomotive and the Co-Co Diesel or Co-Co electric or, in its more exaggerated form, the Bo-Bo electric; and one could look to the development of the Bo-Bo rather than the Co-Co. With the steam locomotive, at the time when the heaviest axles were over a particular point, creating their maximum normal bending moment, there would be an array of lesser loads creating a negative bending moment at that point. In bogie stock that effect was absent, and in fact the relatively high bending stresses produced by bogie locomotives were duplicated on a smaller scale with coaching stock.

Dr T. P. O'Sullivan (Partner, T. P. O'Sullivan & Partners, Consulting Engineers) referred to the question of skew bridges and suggested that the effect of a skew was very small unless the skew exceeded 20°. In the design of a skew bridge a number of factors were involved, such as the angle of skew and the ratio of width to span. In a case for which he had recently been responsible, precast prestressed units had been used, with transverse prestressing; in such a case it would have been possible to design the deck with the beams in line with the parapets, in which case the advantage of the square span would be lost and one would design for a bending moment about 50% more than the square-span bending moment. There was a proposal in a Ministry of Transport publication that in the case of skew bridges of an isotropic nature that they be designed on the square span, in which case the outer girder would be designed for its own load and also to take the triangular one. Such an arrangement, however, might encroach on valuable headroom.

194. It had been found in the case to which he referred that by pursuing a rigorous and so-called exact method of design the critical design bending moment was about five-sixths of the bending moment taken on the skew span. It was about half-way

between the designs for the span measured square to the supports and along the line of parapets.

195. Finally, he wished to make a plea in connexion with skew bridges to the Ministry of Transport. Since the rigorous design was most involved he wondered if it would be possible for the Ministry, in one of its publications, to formulate an appendix which would incorporate certain tables allowing the design of skew bridges to be simplified to the greatest possible extent.

196. On the question of whether with prestressed bridges solid construction should be used or a void lift inside or between the decking units, he had found that up to a span of 40 ft it was advantageous to keep it solid, but over 50 ft it was better to leave spaces between or inside the prestressed beams.

The following contributions were received in writing.

Mr Eric Davison (Assistant to Technical Manager, Lafarge Aluminous Cement Co. Ltd) asked what type of cement had been used for the quick-setting concrete placed in the column gap (§ 34).

198. In § 43 the Author had mentioned a search for cement which would resist acid attack. Had high-alumina cement been considered and if so why had it been rejected as unsuitable? Mr Davison thought that high-alumina cement would probably have been satisfactory from both points of view—quick-setting and acid-resistant. The Author would know that high-alumina cement hardened rapidly and attained high strength at an early age. It also possessed acid-resisting properties although it was not an acid-proof cement. For example, it was resistant to many dilute acids found in the chemical industry, particularly dilute sulphuric acid of concentration equivalent to pH-values of 3/4, and in this connexion it was used extensively for linings in chimneys where conditions were similar to those found in tunnels and beneath underbridges.

Mr T. N. C. Bulman (with Sir William Halcrow & Partners) referred to the very high cost, probably millions of pounds, of raising thousands of overbridge soffits to accommodate the overhead power lines. Would it not be possible to omit those power lines under the bridges (or perhaps have dummy lines) and so to space the pantographs along the crest of a train so that only one is out of contact at any one time? This seemed to be the principle adopted at cross-overs and similar places when power was supplied from a third or fourth rail. The consequent reduction in headroom requirements might eliminate the need for reconstruction work in some cases and certainly reduce it in others.

Mr A. D. Holland (Engineer (Civil), Ministry of Transport and Civil Aviation) noted that in § 50 the Author urged acceptance of substantial tensile stress in the soffit of prestressed members under full live load. If full live load meant Ministry of Transport standard loading, many bridges might be expected to be subjected to this loading on countless occasions during their lifetime and, less frequently, to considerable overload. This design loading was in a rather different category from the one considered in § 48.

201. As the Author knew, the requirement that no tensile stress should occur under working load in bridges subject to impact, was supported by the recommendation to this effect contained in the First Report on Prestressed Concrete published in September 1951. It was also in accordance with general design practice in the United Kingdom.

202. While prestressed concrete was in the development stage the application of this requirement could, in the generality of cases, be amply justified. In prestressed concrete the zero stress, to which a calculated tensile stress was related, was governed by many factors. Due to all or any of these, reality might not accord with calculation. Considerable discrepancy could occur because of inadequate allowance for losses. The

allowance now thought appropriate was considerably higher than that used only a few years ago.

203. It was possible therefore that any theoretical tensile stress used in past designs might, in practice, have been substantially exceeded. Where, however, designs had provided for a theoretical zero stress under working load the concrete strength had been available to accommodate any unforeseen tension that had occurred.

204. Dr Hajnal-Kónyi had referred (§ 163) to an earlier statement[11] regarding provision for transverse moment in slab decks. The approximate values given for this moment had since been confirmed by many loading tests both in the laboratory and on bridges in service. The point at issue was how this transverse moment should be provided for. It was generally accepted that for decks consisting entirely of individual units placed side by side the best method was to prestress transversely.

205. The Ministry of Transport had not insisted that transverse prestress should be used in composite slab bridges. The transverse characteristics, however, could not be ignored, and, as an interim measure, the same treatment had been followed as that used for many years in reinforced concrete bridge decks.

206. In this connexion, Fig. 8 showed transverse steel consisting of $\frac{5}{8}$-in. bars at 2-ft centres. Would the Author confirm that in bridges subject to Ministry scrutiny, an amount in conformity with reinforced concrete slab design was in fact provided at the Ministry's request?

207. In most slab decks the stresses due to the transverse moment when calculated on a homogeneous section were small, and seldom in excess of 400 to 500 lb/sq. in. With a good-quality concrete infill, these stresses were below the modulus of rupture, although the amount of overload needed to cause cracking might be small.

208. Recent tests had been reported[14] by the Cement and Concrete Association on a composite slab deck having a small amount of transverse reinforcement, the amount being limited to that needed to satisfy ultimate load requirements for the deck as a whole. These tests had shown that the deterioration in distribution characteristics of this deck was of limited extent within a considerable range of overload beyond the point at which cracking occurred between adjacent beams.

209. This experimental evidence provided an answer to the point raised by Mr Holland in the discussion on a Paper[17] presented at the Symposium on the Strength of Concrete Structures held in May 1956. It seemed, however, that if ultimate load methods of design were used when considering the transverse strength of bridge slabs, the effect of this in the longitudinal direction should not be overlooked.

Mr A. W. Hill (Deputy Director of Research and Technical Services, Cement and Concrete Association) remarked that in all cases the decision as to whether a bridge or other structure should be reconstructed, or altered or strengthened in one way or another, called for careful consideration of performance, initial cost, and subsequent maintenance. With any new structure the economy in initial cost had to be carefully adjusted in relation to the cost of subsequent maintenance. The prestressed concrete masts described in §§ 86, 87 offered a substantial saving in maintenance cost as compared with the steel masts, while the initial cost and the cost of erection would appear to be similar to those for steel.

211. It had been stated that the concrete mast was heavier, but the difference was so small that the same lifting equipment would deal with either type and the speed of erection would be similar.

212. The tests had shown the ability of the concrete masts to withstand impact when untensioned steel was incorporated in the usual manner. The Author also appreciated the advantages of using a circular section, for which the spinning process was so suitable, and many examples could be quoted from the Continent. In the interests of efficiency and economy it was to be hoped that masts of this type would be incorporated in the British modernization programme also.

The Author, in reply, agreed with Mr Bridgman about the weights of solid decks being excessive in the longer spans. A length of 50 ft had been considered as the change-over point between solid and hollow decks, though, on score of cost, the quantity of beams to be manufactured was another factor, together with plant availability for handling. In this respect the Commission were at an advantage in that they possessed heavy breakdown cranes as a matter of course. That being so, spans of up to 70 ft could be considered for manufacture away from the site.

214. The spans of all bridges referred to in § 135 by Mr Bridgman were about 30 ft and the loading were M.O.T. standard. So far as costs were concerned, these had recently reverted to approximately £12 10*s*/sq. yd of deck and were not, therefore, related to any general price level. At £12 10*s* the Commission were obtaining a very reasonably priced bridge.

215. The Author considered Mr Rosbotham's assessment of the prestressed masts too pessimistic. The Author did not agree that erection difficulties had arisen on the Southend line and, moreover, maintenance costs alone would ultimately dictate the design of masts and bridge members. In this respect, prestressed concrete would undoubtedly come into prominence.

216. Turning to Mr Cockcroft's comments, the Author could not recollect any case on the Eastern Region of British Railways where alterations to approaches, coupled with the use of deck of average depth had proved to be the cheapest solution, and furthermore, the raising of road levels could require the obtaining of Parliamentary Powers. All efforts had, therefore, been directed to solutions involving shallow decks and it must not be overlooked that negotiations with highway authorities, already far too protracted in a number of cases, must, perforce, become even more so if approaches were altered as well.

217. In the more rural areas, such as appeared to have been the case in the north-eastern area, the difficulties were, however, generally easier to overcome.

218. Mr Cockcroft had referred in § 167 to Figs 5 and 9, and had assumed that a third of the total construction depth had been "wasted". These Figures indicated sections at the centre of the bridge, and a certain minimum camber in the roadway was required. In view of the simplicity of execution, it was not advisable to form the bridge deck with a camber, though this had been done in a few cases. The soffit level shown at the centre of the Figures mentioned was required to accommodate overhead line equipment. It would have been preferable to maintain the soffit level which was permissible below the footpaths throughout, but this was not practicable on account of electrical equipment which passed under the bridge.

219. With regard to vibration, mentioned in § 168, Mr Cockcroft was under a misapprehension. The resonance trouble which had occurred at Liège had, in principle, no relation whatever to railway or road bridges. Resonance would not occur in these bridges unless a dangerous frequency was maintained for an appreciable time.

220. Resonance occurred at the tests after the dynamic loading of 250 c/min had continued over a considerable time, but not in individual short instances. It should not be overlooked that the damping effect of following loads would assist in keeping resonance in check.

221. Mr Jenkins had mentioned £20/sq. yd as the price of a deck having a depth/span ratio of 1/30 and constructed with filler joists. The "wafer" deck cost approximately the same price at Ipswich Road, Colchester, but that had been an isolated instance, and there would no doubt be a considerable price reduction when considered in quantity, and, of course, there was a very great saving in the quantity of steel with the "wafer" design. The filler-joist design would, undoubtedly, engender hair cracks upon the soffit of the slab, which, unlike the case of prestressed concrete, must remain open perpetually.

222. Mr Bramall would no doubt refer to Mr Campbell's contribution in § 117.

223. The Author supported Dr O'Sullivan in his plea in § 195, and his point about solid decks and those with voids was covered in the reply to Mr Bridgman.

224. The Author advised Mr Davison that he had not obtained details of the quick-setting cement used in the column gaps at Ars. So far as the use of aluminous cement was concerned, this had been incorporated in a number of prestressed beams in special ocations. In bridge works, however, where considerable humidity and high temperature could obtain, it had been felt that low-heat metallurgical super-sulphated cement was the more suitable.

225. Mr Bulman's suggestion was one that had exercised the minds of many engineers. Dead sections of overhead contact wire were provided in special cases, but these required jumper cables and section insulators, which, at the moment, were not available for use at the highest speeds. The dead section was kept to the shortest possible length.

226. The provision of a third or fourth rail on overhead systems was not recommended, since all motor units must perforce have shoes fitted as well and automatic switching of some kind would be required to obtain complete isolation of the pantograph at a critical moment. Reduced voltage was preferable to dead sections and this method was intended to be applied in some built-up areas and in tight tunnels located within high-voltage areas.

227. To Mr Holland, the Author would suggest that the incidence of loading, even on highways, was such that full M.O.T. loading on the basis used in an assessment for design was only intermittent, interspersed, as the heavy loads were, with a multitude of lighter vehicles. Because of this, the destructive loads applied in the various successful tests did not occur and Mr Holland was well aware of the results which had been achieved at Liège and elsewhere, even with beams deliberately fractured before undertaking punishing dynamic loadings repeated several million times in rapid succession.

228. Regarding losses, the designs carried out at King's Cross had always taken the fullest account of these and allowances, based on tests in 1949, had generally been well in excess of those adopted elsewhere and were considered to be realistic. The fear expressed by Mr Holland in § 203, was, so far as the King's Cross designs were concerned, not, therefore, thought to be applicable. The Author referred to Fig. 4 in which the original design of 1948 had been compared with an analysis of 1958, based on the conditions of the British Standard Draft Code of Practice. It would be seen that there was a difference of only 30 lb/sq. in. in the tensile stress.

229. So far as § 206 was concerned, the Author would advise Mr Holland that the transverse reinforcement now provided conformed to the arrangements made recently with the Ministry of Transport and Civil Aviation. However, the transverse reinforcement in the Great North Road Bridge at Stamford (see Fig. 9) was only four $\frac{5}{8}$-in.-dia. bars in the whole deck and the test with the 90-ton bogie bore witness to what could be achieved in a suitably designed composite type of deck with the minimum of transverse reinforcement.

230. The stresses due to transverse moments, given in § 207, were arbitrary and longitudinal cracks have not been noticed at the tests mentioned in § 208 at an overloading amounting to $1\frac{1}{2}$ times the abnormal loading of a bogie of 90 tons, which in itself was already an overload as compared with the standard M.O.T. loading. Thus the tests have shown that a small transverse reinforcement was enough to ensure a satisfactory load factor, based upon the combined effect in longitudinal and transverse directions, as Mr Holland seemed to suggest in § 209.

REFERENCES

3. T. J. Gueritte, "Further data concerning prestressed concrete: comparison between calculated stresses and stresses registered during test". Correspondence by P. W. Abeles, J. Instn civ. Engrs, vol. 16, p. 520 (Oct. 1941) and Table VIII on p. 529.
4. *Ibid.*, p. 548, line 11.
5. A. A. Paul, "The use of precast prestressed concrete beams in bridge deck construction". J. Instn civ. Engrs, vol. 21, p. 19 (Nov. 1943).
6. *Ibid.*, vol. 22, p. 365 (Oct. Suppl., 1944).
7. *Loc. cit.*, p. 372.

8. F. G. Thomas and Andrew Short, "A laboratory investigation of some bridge-deck systems". Proc. Instn civ. Engrs, pt. I, vol. 1, p. 125 (Mar. 1952).
9. *Loc. cit.*, p. 181.
10. *Loc. cit.*, p. 185.
11. A. D. Holland, "Prestressed units for short-span highway bridges". Proc. Instn civ. Engrs, pt II, vol. 4, p. 224 (June 1955).
12. "Test of a prestressed concrete railway bridge". Concr. & Constr. Engng, vol. 46 (1951), p. 186.
13. B. C. Best and R. E. Rowe, "Abnormal loading on composite slab bridges. (1) Tests on a bridge without transverse reinforcement". Tech. Rep. TRA/301, Cement & Concr. Ass., Oct. 1958.
14. B. C. Best and R. W. Rowe, "Abnormal loading on composite slab bridges. (2) Tests on a bridge with some transverse reinforcement". Tech. Rep. TRA/307, Cement & Concr. Ass., Dec. 1958.
15. Francis Walley, "St James's Park Bridge". Proc. Instn civ. Engrs, vol. 12, p. 217 (Feb. 1959).
16. H. I. Andrews, "The adhesion of electric locomotives". Proc. Instn elect. Engrs, pt A, vol. 102a (1956), p. 785. Discussion by F. Whyman, vol. 103, p. 203, and H. Charnley, p. 204.
17. P. B. Morice and G. C. Reynolds, "The strength of simply supported slab bridges subjected to concentrated loads". Proc. Symp. on the Strength of Concrete Structures, 1956. Cement & Concr. Ass. (To be published.)