Museum Radio Tower

by

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The Chairman (Mr Ralph Freeman) said that 12 or 13 years previously his firm had been designing chimneys of 420 ft in height, and had had wind tunnel tests done by the late Dr N. A. V. Piercy. The original design had been for 12-sided chimneys, but it was found that any form of chimney with vertical striations produced regular shedding of vortices alternately from one side and the other at wind speeds within the range under consideration. Dr Piercy had recommended that the chimneys should be plain, as smooth as possible, and circular. Mr Freeman said that he was sure that vortex shedding was connected with the shape of the chimney and the opportunity for the vortex to take off.

Sir Albert Mumford (Engineer-in-Chief, Post Office; Immediate Past President, Institution of Electrical Engineers) said that it had been made clear that the Museum Tower formed part of a microwave network of trunk communications, the main trunk network of which consisted of coaxial cables buried below the surface so that they could not be seen. The Post Office was now engaged in supplementing and complementing this by an extensive network of microwave links. Both systems provided wide-band facilities for all purposes. The Museum Tower was one of a series to be erected at roughly 30-mile intervals.

171. The only minor criticism which he would make of the Paper was that it referred to the Museum Tower as handling 150 000 telephone circuits. It should be added that it would handle simultaneously 40-50 channels relaying television. This gave an idea of the magnitude of the requirement, and the figures related, moreover, only to the present stage of development. Techniques might in time be improved in order to get more out of such a structure.

172. The Museum Tower was a wonderfully imaginative concept for meeting a most difficult requirement. The project seemed to have aroused the interest of engineers and non-engineers who he had met abroad, and he was certain that it had enhanced the reputation of British engineering.

173. He had been glad to hear that the curiously shaped aerials on the tower had seemed to simplify the engineers' task.

174. Finally, he would like to pay his tribute to all who had been engaged on the project.

Mr E. O. Measor (Scott & Wilson, Kirkpatrick & Partners) wished to raise three points on the design of the foundations, with special reference to the bridge link.

This was obviously very useful for controlling the lateral movements of the tower, but it seemed to present some difficulties. He was not clear from the information given in §§ 87–90 on the exact details of the hinges connecting the link to the tower. He imagined that these were straight, running approximately north and south and only hinging the slab for relative settlement between the tower and the adjacent building, and that the connexion of the bridge to the tower was not really articulated. When the tower deflected in the approximately east–west direction this was probably not of very great importance, but when it deflected north–south it seemed that the mechanism was rather complicated and involved tearing forces in the link as the torsion of the building and the bending of the cantilever were opposed by the torsional rigidity of the tower. Had any consideration been given to passing the tower through a hole in the link, with rubber bearings, which would have formed a more complete articulation?

176. From the diagrams showing the behaviour of the tower he gathered that the yielding pins were assumed to have much the same behaviour in all directions; presumably in the north–south direction, where there was torsion of the adjacent building, there was rather more flexibility, though he did not know that this would necessarily be of any great importance, because the calculations of the yield of the yielding pin were probably on a rather by-and-large basis and there would be some margin for variations in behaviour.

177. Secondly, when a connexion was made from a tall tower to an adjacent building by a link, this brought in the question of uneven settlement. London Clay was extremely uniform; nevertheless it was unsound not to make provision for forces which would be induced if unequal settlement occurred. With reference to the diagrams which Mr Creasy had shown, if there was unequal settlement, or at least an unrestrained unequal settlement of, say, 25% of the predicted 20-year total settlement, it would represent about $\frac{1}{2}$ in. This would give approximately similar horizontal movement at the level of the link which would have to be counteracted to some extent by the bridge link acting as a yielding pin, and stresses would be set up. Had consideration been given to the possible forces induced by leaning of the tower, and what view did the Authors take of this? It might be of no great moment in the present design, but it was often of very serious importance where tall towers were attached to some adjacent podium.

178. From the figures given, he would have thought that the foundation could have been enlarged slightly and the link avoided, so that all the accompanying complications would not arise. In the diagram the deflexion was given as $\frac{3}{8}$ in. as compared with a non-restrained tower, so that a slight increase in the foundation would, it seemed to him, have given the same deflexion as that obtained with the restraining link. He was always very much happier about a tower which stood on its own. When it was attached to an adjacent building it became necessary to restrict any possibility of future alterations. In twenty years' time the Post Office might wish to enlarge the building and this might be inconvenient.

179. He was very interested in the section of the Paper on aerodynamic stability (§§ 42–56), as his firm had been concerned with this on a number of structures recently. The Authors referred in § 47 to observed values for the structural damping factor δ , which, they said, might 'lie between 0.03 and 0.2 depending on the structural form and method of support'. As the Authors would be aware, the data on which those figures were based were scanty and rather unsatisfactory, but the factor itself could be of overriding importance. He suggested that the Authors should add to the array of instruments on their building by making at least one attempt to measure the damping factor. Some people had fired off horizontal rockets from their buildings, but this might be unpopular in Bloomsbury and in any case had not proved entirely satisfactory. The use of a delicate accelerometer and a moving weight might enable some observations to be made.

180. There was nothing in the Paper about glazing design, but it was of great

importance in all tall buildings where large sheets of glazing were used. It was a vital part of the design, because the failure of a single sheet at a high altitude could have lethal consequences. The investigations which his firm had made into the strength of the glass used had emphasized that the situation was not at all satisfactory, and that a large amount of testing would have to be done to give convincing results. In his view the chances against failure of a single sheet should be extremely high. It would be of interest if the Authors would say a little more about glazing design and the view which they took of what theoretical chance of the failure of a single sheet would be acceptable.

181. In connexion with the design of basement slabs, he was interested in the slip joint of the plastic-sheet sandwich with oil. He was always sceptical about the correspondence between small-scale tests and the behaviour of a large joint under a very large slab, with the possible effect of surface roughness. What coefficient of friction had the Authors assumed for the design?

Mr G. W. Kirkland (Travers, Morgan & Partners) said that he wished to seek elucidation on a few small points.

183. With regard to vortex shedding, his own researches into tall chimneys had shown that on the leeward side of a circular structure a vortex invariably formed and was shed when equilibrium ceased to be maintained. He wondered whether the tests carried out at Teddington indicated anything of that nature. He was aware that the test had shown a lateral oscillation, which was now beginning to be better understood by engineers, but no reference had been made to the vortex phenomenon to which he referred. It might well be that the horn-shaped aerials helped to cause surface disturbance by breaking down what would otherwise be a dangerous vortex.

184. He would also like to know whether the Authors had investigated the effect on the overall complex of buildings in the vicinity due to the construction of a high tower. He had taken observations recently in connexion with two other tall structures with which he was involved and had found that it was necessary to carry out certain researches to determine the environmental effect. In one case this was part of a complex hospital development with certain covered ways for the use of ambulances. He was concerned from experience lest ambulances entering a tunnel should find themselves in the equivalent of a Lamson tube and become sucked to an unintended destination leaving their passengers on the pavement.

Mr G. J. Zunz (Partner, Ove Arup & Partners) said that about four years previously, when he had been concerned with the design of a radio tower in South Africa,¹⁰ of similar dimensions to the Museum Tower, a very interesting letter had appeared in the local press. It had been written at a time when the construction of the tower had advanced to about 60 ft above ground level, and it had been on these lines:

'Dear Sir,

I have watched with considerable consternation the commencement of construction of the radio tower. My house is situated very near the tower, and I have therefore measured the distance from its base to my house. To my relief I find that when the tower collapses it will miss my house. However, could you ascertain from the experts if there is any danger of it rolling?

Yours,
etc.'

His own interest in the Museum Tower was on the same lines, because he spent most of his working life in its shadow; but after reading the Authors' excellent Paper he was somewhat reassured.

186. The apparent simplicity and grace of the Museum and other towers really belied the very complex technical problems facing the designer. The design and conception were quite formidable problems, and it was not surprising, therefore, that the Authors had begged a number of vital questions. A great deal of practical and

theoretical research was required before it was possible to approach the design of radio towers, television masts, and the like with the same degree of comfort as most other structural engineering problems. The courage of the Authors in publishing their work and affording an opportunity for discussion was wholly commendable.

187. When designing tall buildings engineers examined the wind forces in some detail, but more often than not the effect of forces other than these was equally if not more significant. Moreover, because of the need in most buildings to provide more structure than was necessary to resist the forces of wind alone, the restraints produced by those additional structural members often ensured safety even if the wind forces themselves were underestimated. When examining the structural adequacy of the Museum or any other radio tower, however, it would be found to be largely dependent on the choice of wind velocity, gust duration, aerodynamic stability, etc.

188. With regard to wind velocity, the Authors had shown their design considerations, and one of his colleagues would describe how the velocity profile for the South African tower had been chosen, taking wind speeds somewhat in excess of those adopted by the Authors. The importance of this could not be over-emphasized, because an increase in velocity from 72 to 90 mile/h increased the pressures by 56%. However, the choice of the velocity profile was not exclusive and the whole design philosophy had to be considered.

189. He would like more information on the reasons for the choice of a wind velocity using a theoretical period of 50 years. This had a 2% chance of being exceeded in any one year and, since the pressure and hence the overturning moments increased as the square of the velocity, to underestimate the velocity was a serious matter. Furthermore, the gust duration, the choice of a 2-, 10- or 15-sec gust, should be related to the structural response. Did the Authors consider this response when choosing their design pressures?

190. Having attempted to establish the nature of the wind forces, the next problem was to find the actual forces on the structure, having regard to its shape and size. This type of structure should in the first place be designed for ultimate load considerations. It was a matter for speculation and discussion what ultimate load factor to use and whether it should be based on wind velocity or wind pressure. Had the Authors considered this problem of ultimate load? What load factors did they use and on what were they based? The South African tower to which he had referred previously was designed for a load factor of 2 based on a wind pressure profile somewhat in excess of that used by the Authors. The choice of ultimate load factor should be based on statistical investigations which were required before detailed proposals could be more than good intuitive guesses.

191. Why had the Authors chosen a drag coefficient C_d of 0.75 for the apparatus and the shaft section? Did they think that the faceting of the tower did not substantially increase this coefficient? Had they carried out any tests to confirm their choice?

192. The aerodynamic stability of these towers was probably even more difficult to predict with the present state of knowledge. Engineers were concerned mainly with two types of oscillation: transverse oscillation due to vortex shedding and oscillation in the wind direction due to periodic gusts. Very little was known about oscillations of the latter type and though they were thought to be of less importance than the transverse oscillations, towers of greater height and slenderness and lower frequencies could create situations where this problem required serious investigation.

193. Tests on models carried out in wind tunnels made it possible to predict the degree of aerodynamic excitation and damping of particular shapes. It was clear from recent investigations that even at the higher super-critical Reynolds numbers periodic shedding of vortices took place, a phenomenon once associated only with sub-critical Reynolds numbers. It seemed, therefore, that *all* circular shapes would be positively excited over a range of wind speeds likely to be encountered in practice. While aerodynamic damping of lattice masts and such open sections on the Museum

Tower where aerials were placed could be inferred from wind tunnel tests, neither the extent of structural damping nor the precise value of the log. decrement was known. Damping could well vary during the life of the structure and reinforced concrete might have a varying decrement in various stages of its life, when it was cracked or uncracked. More information was needed on structural damping constants and he would like to have graphs or tables taking account of the various parameters to enable prediction of structural damping to be made with some degree of certainty. This was very important, because materials were being used more efficiently and new ones invented and used under higher stresses. A situation very like that which had arisen with some earlier suspension bridges due to dynamic instability might follow.

194. With regard to function and aesthetics, it was always very interesting to speculate on the aesthetic consequences of a purely functional rationalization. It was a topic discussed endlessly by engineers and architects. If he had any criticism of the Museum Tower, it would be that it did not go far enough to emphasize its function. The bare structure with its cantilevers still open at the top was marvellous, but he wondered whether more delicately handled cladding detail could not have retained its essential structural character.

Mr W. W. Frischmann (C. J. Pell and Partners) congratulated the Authors on their excellent Paper, and expressed his conviction that the Museum Tower would rank with Big Ben as a London landmark. Aesthetically he found it a very pleasing building indeed, but there were a few points which slightly disturbed him, particularly the bridge link. To his mind, a very pure structural form had been slightly spoiled by this bridge. It seemed obvious to him that it was the result of an afterthought by the Client. He felt that it would have been better to make the foundations a few inches wider, and in this way the need for a bridge would not have arisen. A very simple form, the simplest of all natural structural forms, seemed to be stabilized by a bridge action which was very complex indeed.

196. He had tried to analyse the benefits of the bridge, and had studied what was said in the Paper. It seemed to him that the gain was very small. He could, appreciate the point that the bridge must allow for vertical movement in the building, and therefore a hinge had to be produced in two planes. In order to prevent tilting, the other end of the bridge had to have another hinge, and so there were basically two hinges which in space could not act as such, since they had to stabilize the tower by torsion in one direction. The stresses in these members were very high indeed, and he wondered about possible fatigue in the reinforcing bars. With regard to the foundations (Fig. 4), it was stated that the clay varied by as much as 5 ft from north to south. Because of the very high compressibility of the clay, he would have thought that there might be a tendency for the tower to tilt. He wondered whether there was any device in the base of the tower to counteract this, and if some correction had been made after the building had been completed.

197. He was concerned with some tall buildings, and would like to know the speed of the lift serving the public areas, since the great height served by the lifts without any intermediate stop could have allowed a really fast lift.

198. With regard to aerodynamic stability, he was uneasy about some points in Fig. 12. It was shown that there was considerable difference in behaviour between the sheeted aerial section and the unclad aerial section. It seemed that the model had been tested for both cases, and there seemed to be a much larger difference between these than between the typical unstable trace and the aerial section used on the building. What would be the effect if a change were introduced into this particular part of the building? Future development might require covering over completely, and he thought it possible that the building would show very dangerous signs of aerodynamic instability.

199. Obviously, damping of structure was very important, and he wondered what damping factor had been used to convert the model results to concrete buildings.

200. Had the Authors considered the possibility of hanging the floors from a cantilever arm? In a number of buildings serious consideration had been given to placing a cantilever arm at the top of the building and hanging the floors from this. In particular, the dead weight of the floor would bring the load almost to the top of the building, and this would have the effect of prestressing the tower, with obvious benefits.

201. The comparatively low height of the tower was disturbing. There were several buildings in course of completion which would be higher than the proposed tower aeriels, and he wondered whether the tower building might not become obsolescent before it had been completed. He had tried to check the legal position and believed that the erection of a building higher than the Museum Tower would be permitted. However, would it be necessary to get the permission of the G.P.O. to erect a building which would be higher than the existing aeriels?

Mr D. J. Lowes (Senior Engineer, Ove Arup & Partners) said that the present position regarding the estimation of wind forces remained notoriously uncertain. In many circumstances a designer was obliged to assess the wind loading and decide upon appropriate ultimate load factors for a structure when there was relatively little information or precedent available to guide him. Mr Lowes had met with these problems at the time when he worked on the design of the Brixton Radio Tower in Johannesburg, the project to which Mr Zunz had referred. There was no doubt that designers in the future would be grateful to those responsible for the instrumentation of the Museum Radio Tower for the basic data that would be collected.

203. There seemed to be a case for a conservative estimate of wind loads coupled with the choice of moderate ultimate load factors. When there was controversy about which wind-speed profile was meteorologically nearest the truth, then the least favourable profile in terms of loading was probably the most suitable for stress and stability calculations. Fig. 44 showed profiles that the Authors mentioned compared with others based on different assumptions.

204. In Fig. 44 the contributor's profiles were referred to a height above ground of 33 ft instead of the 133 ft suggested by the Authors. This was because it had become customary in the literature of the subject to use approximately 33 ft as a reference height. The effects of the terrain 'roughness', such as buildings, could then be represented solely by the value of the exponent α . The values of α which were regarded as representative of different wind-speed profiles could only be compared provided that the other characteristics of the profiles, such as the reference height, were kept the same. A profile with a mean hourly maximum wind speed of 33 mile/h at 33 ft and $\alpha=0.25$ was much the same as the Authors' mean hourly profile and it avoided the introduction of the additional concept, 'effective ground level in London'. It also indicated that what might be understood as the corresponding maximum mean hourly 50-year wind speed at reference height 33 ft, namely 33 mile/h, was a little on the low side.

205. Another profile was drawn with $\alpha=0.17$ to show that for the same exponent a lower reference height would lead to slightly larger forces being calculated, but the difference was not very important. The max. mean hourly 50-year wind speed of 43 mile/h at 33 ft was certainly a reasonable value.

206. Both Shellard¹¹ and Davenport⁴ had suggested that for south-east England the max. mean 1-hour wind speed with a return period of 50 years was something a little over 100 mile/h at gradient height. That height was said to lie between 1000 ft and 2000 ft, depending on the terrain. The exponent $\alpha=0.17$ for the max. mean 1-hour 50-year wind appeared to be too small because it was inconsistent with this gradient wind speed. In Mr Lowes' curve with $\alpha=0.17$, the calculated values of wind speed at 1000 ft and 2000 ft were 77 and 86 mile/h respectively. The Authors' comment that buildings retarded wind speed near the ground was accepted, but this was with reference to the gradient wind speed. If the profile was being worked out

FIG. 44: WIND-SPEED PROFILES

from low-level anemograph records pertaining to a city, then the corresponding exponent must also be used—the larger the more conservative. Shellard had suggested the exponent $\alpha=0.23$ for London and mentioned that measurements on the Museum Tower would help to confirm or refute this figure. The curve with $\alpha=0.25$ appeared to be fairly reasonable. The corresponding theoretical wind speeds at 1000 ft and 2000 ft were 101 and 120 mile/h respectively.

207. Shellard had suggested ratios from 2.0 to 2.2 to relate the highest gust speeds in city centres to the max. mean 1-hour 50-year wind. Using the higher figure, the maximum gust speed would be 95 mile/h at 33 ft (133 ft in the Authors' terms). Information about gust factors and their variations with height remained extremely sparse. Scruton and Newberry,¹ quoting Nutt, mentioned the working rule of assuming a maximum gust factor equal to 1.25 at gradient height. This figure had been found to be consistent with such gust observations as had been reported, including Deacon's 0.85 used by the present Authors, provided that the terrain was taken into account in each case. The exponent connecting 2.2 at 33 ft and 1.25 at 1500 ft was -0.15 . The corresponding profile exponent when the 1-hour wind was multiplied by the gust factor was $(0.25-0.15)=0.10$. The Authors' and the contributor's extreme gust curves were not very different; the pressure difference at 400 ft would amount to about 10%.

208. The choice of the gust period depended largely on the response time of the structure and its size. The response time was the time it took for the structure to become fully stressed under an applied load. If there was no damping the response time of the structure would be about one quarter of its natural period of oscillation. In the case of the Museum Tower this would mean that the tower could be fully stressed by any load lasting longer than $1\frac{1}{2}$ sec. According to Whittingham¹² the minimum effective gust must have sufficient vertical and horizontal extent to envelop not only the structure but also those flow patterns to the windward and leeward which were responsible for the maximum pressures on the structure. The length of the gust needed to be about eight times the width of the structure, a length of about 400 ft in the case of the Museum Tower. A gust at 100 mile/h could therefore envelop the flow patterns in less than 3 sec. A gust acting on the top 200 ft of the tower would produce about 80% of the bending moment about the bridge level that it would cause if it acted over the whole height of the structure. It was therefore reasonable to assume the maximum gust speed for estimating the design loading. Comparing his maximum gust speed with the Authors' 15-sec gust, the corresponding

increase in pressure would be about 85% and the increase in bending moment about the bridge, allowing for a gust acting only on the top 200 ft, would be about 50%.

209. Fig. 44 referred only to wind speeds with a return period of 50 years. The probability of the 50-year wind speed being exceeded one or more times in 100 years was 87%. If the design load were based on the 50-year wind, then that load would very likely be exceeded during 100 years. Therefore it was reasonable to consider a wind speed with a greater return period. The wind speed with a return period of 200 years had a probability of 40% of occurring during a period of 100 years but it was also at about the sensible limit of extrapolation from available records. The probability of the statutory design-imposed loadings on buildings actually being realized was considerably less than 40% in most circumstances. Nevertheless, because it was a fairly reliable estimate, the 200-year wind speed was a suitable figure upon which to base the assessment of wind loads. Although it was an unreliable value, by further extrapolation it was noted that the 2000-year wind speed would account for an increase of 33% in the stagnation pressure over the 200-year wind, and it would have a 5% chance of occurring during 100 years. These probability figures would influence the choice of ultimate load factors.

210. Considerations of the appropriate reference height, the exponent α , the duration of the gust and the return period of the wind all showed that, based on current literature, it was possible to arrive at higher estimates of wind loads than those mentioned by the Authors. However, this was not necessarily significant because in the end it was the relationship between the assumed design load and the ultimate load factors that was most important. Would the Authors enlarge on their approach to this problem in their reply? The information in Table 4 of the Paper was perhaps by itself a little misleading because it gave the impression that structural safety could be assured by limiting the concrete and steel stresses to their ordinary permissible values under the assumed design wind load. This was not so in structures like the tower mainly because the vertical load remained sensibly constant while the wind load increased, and with increase in the wind load the stresses would increase disproportionately rapidly.

211. Fig. 45 was prepared to illustrate the possible behaviour of a reinforced concrete annulus, subject to axial compression and bending. Of course, the annulus in the illustration was not necessarily representative of the tower because it was not possible for the Authors to have put all the relevant information in the Paper. However, as far as could be ascertained, the dimensions and the loadings were the same as those at level 205 ft in the tower. The quantity and quality of the reinforcement were guessed at 0.8% mild steel. The wind load factor equal to 1.0 corresponded to a bending moment of 77 000 K ft due to the Authors' 15-sec gust. The assumed vertical load was 9500 K, inferred from the eccentricity mentioned in Table 4 of the Paper.

212. In Fig. 45 curves were drawn to show the relationship between the wind bending moments (represented by the wind load factor) and the stresses in steel and concrete according to three different theories:

- (i) assuming an uncracked section and modular ratio 6,
- (ii) assuming a cracked section with straight line stress distribution and modular ratio 15,
- (iii) assuming a cracked section with a straight line stress distribution except for all the reinforcement within a 60° arc on the tension side which was assumed to have the same stress, and modular ratio 10.

The collapse moment was worked out assuming that the concrete strain was limited to 0.3%, that all the concrete in the compression zone had reached a stress of $2/3 \times 5690$ lbf/sq. in. and that all the steel where the strain exceeded 0.13% was at the yield stress of 40 000 lbf/sq. in. It was assumed that the 0.8% reinforcement was only stressed by the vertical load and by the wind bending moment, not by any local bending stresses such as those mentioned in Table 4.

FIG. 45: RELATIONSHIP BETWEEN STEEL AND CONCRETE STRESSES AND WIND LOAD FACTOR

213. The theory assuming all the reinforcement in a 60° arc at the same stress was suggested to define a limiting bending moment for permanent set. If the yield stress were ever produced in a 60° arc of the reinforcement, then after the removal of the load the shaft would be left with a small permanent curvature. This was, of course, a rather arbitrary criterion. According to Fig. 45, permanent set would occur at wind load factor 1.7. However, with respect to the max. 200-year gust, permanent set would occur at a load factor of 0.8. In circumstances of very good workmanship, refined structural analysis, wind loads based on a reliable estimate of the 200-year wind speed, and none too serious social or economic consequences of permanent set, an appropriate load factor was more like 1.3 than 0.8, based on the 200-year max. gust.

214. It was difficult to ascribe any particular meaning to the collapse moment calculated by ordinary methods when it was known that the largest loads would only act for a very short time. However, because it was common practice to apply wind loads as if they were statically applied loads, this was regarded as a reasonable reference strength. Fig. 45 showed that by ordinary standards collapse would be anticipated at wind load factors of 2.2 with respect to the Authors' 15-sec gust and a little less than 1.0 with respect to the contributor's 200-year max. gust. It was felt that in a structure like the tower, on the same assumptions regarding workmanship, wind load estimates and analysis as mentioned before, and taking into account the consequences of collapse, an ultimate wind load factor of 2.2 with respect to the 200-year max. gust was possibly a reasonable value.

215. Was there any similarity between the behaviour of the annulus as described in Fig. 45 and the expected behaviour of the tower section at the 205 ft level; was the tower substantially different by reason of different loadings, dimensions, reinforcement or other characteristics, or was the behaviour of the tower more satisfactorily interpreted by other methods of analysis? What were typical values of the ultimate wind load factor at other levels in the structure?

216. With regard to lightning, the Authors mentioned that provision had been made for conducting electricity from a lightning flash down the reinforcement. Was there any danger of damaging the structure by this device?

217. Would the Authors comment on the relative merits of the kind of base that had been used for the Museum Tower, and one made from solid concrete?

218. Experience with the use of bentonite on a project with which Mr Lowes had been associated, had shown that the bond strength of reinforcement was impaired when it was coated in bentonite slurry. It was desirable to have some mechanical anchorage such as hooks at the ends of reinforcing bars in such circumstances. What were the Authors' views from their experiences?

Mr D. H. Little (Superintending Engineer, Ministry of Public Building and Works) said that he wished to refer to the teamwork, and to say that in the case of the building under discussion the biggest single contribution had perhaps been that of the contractor, but to avoid being controversial he would bracket together the civil engineer in design work and the contractor. The mechanics of the revolving platform were brilliantly simple, but it would be fair to say that the mechanical work occupied the third place. The architectural side was essentially advisory, and he put that fourth.

220. Mr Little fully agreed with Mr Creasy that teamwork was very important, but he also thought that the choice of captain of the team was vital to its success.

221. There had been much discussion of the effect of wind on the tower. Undoubtedly with a structure which soared up 600 ft the effect of wind was the first thing one wanted to know. The towers of the Forth Bridge were 300 ft high, and in 1940 600-ft towers had been designed. In the past few years half-a-dozen towers of this height had been built for the Admiralty and many for the B.B.C. The information which had thus become available ought to be collected so that there could be something really authoritative for future designers. The great value of the present Paper was that it marked a start in this connexion. Possibly the reason why this had not

been done was that it was so often possible to beg these issues, as indeed the present Authors had done in their foundation design. The foundation was 90 ft square and carried a dead load of 13 000 tons. This gave a uniform load of 1.7 ton/sq. ft, which, with the overturning moment, gave a maximum stress of 2.3. If the wind were doubled, the maximum figure would still be only 3, and if it were halved the maximum would be 2. Hence, so far as the foundation was concerned, there could be a very wide range in the assessment of wind load, but the pressure on the ground only varied between 2 and 3 ton/sq. ft. The clay had a shear strength of 1.5 ton/sq. ft, and was 22 ft below the ground. It could safely be loaded to 5 ton/sq. ft, so that if one did not know what the wind load was, it did not really matter. He would like to pay the Authors the very genuine compliment that this foundation was thoroughly worthy of the engineering department of the Admiralty. His only criticism of it was that he would have made it 100 ft square.

Mr G. L. Hargreaves (Ministry of Public Building and Works) observed that most of those who had taken part in the discussion had put emphasis on the significant point, which was windage. He had in the past been concerned with many of the towers that Mr Little had had a hand in designing. The first of the towers with which he could claim acquaintance had been a wartime job, built of steel sections, and the windage calculations had been straightforward. A table of lb/sq. ft of area, which varied with height, had been applied to the sections and a stress diagram had been produced. These towers were still standing.

223. More recently he had been concerned with the erection of three 600-ft masts in a hurricane area. They had been designed for a wind speed of a little over 100 mile/h, and had just been completed when the area received its worst ever hurricane. The recorded wind speeds had been very much in excess of those for which the towers had been designed, and the design had been re-examined. This had been extremely complicated, because after about 25 years of progress the calculations had become so complicated that they could be worked out only by a computer, and when the results had emerged they could be understood by only a very limited number of people, of whom he had not been one. He had, however, made it his business to look at them, and had been taken aback to discover that these extremely complicated calculations depended for their accuracy on a considerable number of assumptions and coefficients, which had destroyed much of his faith in their accuracy. He had done an approximate calculation on a much simpler system and had been happy to find that according to this the masts would stand; they were still standing.

224. He thought that the Authors had had similar difficulties, since a brief examination of the Paper showed that in §§ 30–41 the words ‘assumed’ or ‘assumption’ occurred four times, ‘suggested answer’ twice, ‘probability’ once, and that the wind coefficient varied in extreme cases from 0.7 to 1.3. Further, the Authors made the significant comment in § 36 that ‘the choice of effective gust period is one of the most critical factors in the design of the tower and one of the most imprecise’. This confirmed the impression which he had formed several years earlier.

225. More information was needed on which to base these calculations. He noted with satisfaction and pleasure that the Authors had incorporated an impressive array of instruments into the tower, and he hoped that they would present another Paper in a year or two which would take the question much further. In the past there had been too much concern with factors and coefficients; there was now an opportunity to get real answers. His only other point was that the selected design gust was 15 sec. If it had been 10 sec it would have meant an increase of about 30% in the wind loading.

Mr F. E. Prior (Peter Lind & Co. Ltd) felt that some supplementary remarks from the point of view of the main contractor might be of interest. He said that at the tender stage, the use of continuously-moving forms for the shaft walls had been

considered by his firm but had been rejected because of the sharp taper at the quarter height, the stepped reduction in wall thickness, the accuracy required, the difficulty of providing suitable connexions at the junctions with the floors and at other points of complexity, the density of steel reinforcement, and the welding of this for electrical continuity. On the other hand, they had been unable to visualize completion within the contract time by normal building methods, i.e. by constructing in alternate sequence first 13 ft of shaft, including the building-in of heavy long splice-bars for the floors, followed by the construction of the floor, immediately above this. This method was open to numerous objections, such as the necessity to retain propping for a number of floors in order to distribute the weight of the new floor safely, the difficulty of striking and passing the wall forms outwards through the propping before lifting to their new positions, the possibility that external scaffolding would be required for the full height, with consequent problems of temporary works design, the complete absence of working space on which to lay down material temporarily out of use, and, most important of all, the fact that only one construction working site would be in operation at any one time, with consequent limitation of the number of men who could be economically employed.

227. With the consent of the engineers they had therefore used an 'engineering' method. As described in the Paper, this had consisted of the construction of the shaft in advance of the floors by an adaptation of methods normally used for chimney construction. The matured shaft could then be used as a support for the erection of formwork for each external floor without interference with the construction and maturing of the floors below. It had thus been possible to increase the number of work points to some five or six, to increase considerably the number of operatives at any one time, and to separate with the minimum of interference two very different types of construction, each with its own special operatives and formwork.

228. The Linden crane and the two passenger/material hoists had been lifted and extended to keep pace with the shaft construction, thus providing more adequate access to all points of work. The hoists had been protected from the full effect of wind and weather and had been supported by the concrete shaft. These were very important considerations in such a high structure. The light forms for the shaft could be released, lifted by hand, and refixed immediately in their new position; the comparatively large and heavy floor forms could likewise be lifted by the crane from one position to the next without the necessity of temporary storage. The risk of damage to persons and property at ground level by the blowing away of loose timbers or forms in sudden gusts of wind had been substantially reduced by this method. The necessity for continuous external scaffolding had been eliminated.

229. The Authors and their colleagues had given their full co-operation and by altering the design of the floors and the splice-bars to suit the method of construction adopted had greatly facilitated the contractor's operations. His firm considered that the rate of progress achieved by this method of construction had justified its choice. Despite delays due to the severe winter, the 360-ft height of structure between the B.1 and D.1 floors had been completed in 12½ months.

230. It had been their intention to continue the use of this method to the top of the structure, but the latest revision of the requirements for the public section, mentioned by the Authors in § 40 of the Paper, had caused drastic alterations to the shape of the central shaft, the addition of two more floors and the increase in diameter of most of the floors, the restaurant being as a result approximately 65 ft in diameter.

231. Unfortunately the unavoidable complications had made it necessary to revert to more conventional building methods, but by that time they were nearly 500 ft in the air. Despite their best endeavours, the last 105 ft above the D.1 floor had taken nearly 10½ months to complete.

232. In the construction of the tower over a period of some 123 weeks, 26 weeks had been lost from bad weather, mainly due to high winds which interfered with the use of the crane or made working conditions unsafe at such a very exposed elevation.

233. Although they had supplied the maximum amount of hoisting equipment which could be erected inside what was a very small shaft, precise programming and strict supervision of its use had been essential to reduce to a minimum the congestion resulting from the passage of the contractor's men and materials, the necessary attendance on their subcontractors, and on the Post Office engineers and all their subcontractors.

234. He took the opportunity to express the very high regard of the contractors for the very close collaboration given to them by the Authors and all their colleagues and also for the enthusiasm and tenacity of purpose of all concerned in the project, including the supervisory staff and workmen, who often carried on under most difficult and trying conditions.

Dr A. J. Dutt (Professional Officer, Special Structures, Greater London Council) said that he wished to raise the following points on the effect of wind on the tower.

236. It had been mentioned in the Paper that the observed values of the structural damping factor δ lay between 0.03 and 0.2, depending on the structural form and method of support: this range was very wide and the amount of the damping factor which had been assumed while designing the tower was not stated. What amount of damping had been incorporated and had this been verified from the structure? Structural damping would be important in order to limit the deflexion of the structure by gusty winds. As its natural frequency was 0.16 c/s, the critical speed of lateral oscillation would be approximately 30 mile/h. At this speed aerodynamic damping was low.

237. It might be mentioned that the contributor in collaboration with Professor A. W. Hendry of Liverpool University had found the structural damping factor of a simple cantilever structure having a natural frequency of 3.75 c/s to be 0.093, from the record of free vibration of the structure.

238. As the structure was not exactly symmetrical about its vertical axis there was a possibility of coupling between the bending mode and torsional mode of vibration. He would be glad to know if this had been taken into consideration in the design.

239. He wondered if a simple wind tunnel test in a steady windstream was adequate to predict the dynamic behaviour of such a tall and important structure. Perhaps an aerodynamic model could have been tested in the natural wind so that more definite conclusions about its behaviour might have been drawn. As the model was to a 1/67 scale and as the shape of the structure was circular there would be a scale factor. He wondered if the coefficient of drag found from the wind tunnel would be applicable to the prototype.

240. It would be of interest to know the amount of dynamic magnification of deflexion corresponding to the max. mean velocity of a 94 mile/h gust of 15 sec duration, since this had been the basis on which the final design of the structure had been devised. What was the possible dynamic load factor.

241. Would the rotation of a portion of the tower have any effect on the response of the structure to wind, especially in relation to the phenomenon of vortex shedding? Had any tests been done on this point?

242. Observations on the variations of wind velocity with height over cities were limited. The Authors were to be congratulated for making use of whatever data were available. In this connexion the contributor wished to mention that a research project on the subject had been undertaken by Professor A. W. Hendry and himself at the University of Liverpool, using captive balloons. Their observations over Liverpool could be applicable to many cities.

243. Finally, he would like to know the time taken for the design of such a structure, and also for the wind tunnel tests.

Mr D. E. Thorpe (Associate, Scott and Wilson, Kirkpatrick and Partners) referred to the fact that in Table 1 the mean kinetic wind pressure on the public section of

the tower was given as 21.7 lb/sq. ft, and asked for what pressure had the glazing of the public rooms been designed. From the paper by Scruton and Newberry¹ it would seem that extreme gusts over London at 500 ft could reach 110 mile/h velocity, producing a wind pressure of about 31 lb/sq. ft. Coefficients in the Swiss code for smooth surfaces gave a maximum suction of 1.7 times wind pressure, leading to a value of 53 lb/sq. ft, which seemed very high. He wondered whether the glazing and mullions had been designed for such a high value.

245. With regard to the instrumentation of the tower, would an attempt be made to correlate surface pressures with wind velocity at various levels? It would be very valuable if this were done and would help to remove doubts about the inequality of Reynolds number between model and prototype for cylindrical structure. He had certain reservations about the application of the results to the design of tall buildings in general, as seemed to be implied in § 13, since they would be applicable only to those of a cylindrical shape, which was rarely used.

Mr R. E. Whitbread (National Physical Laboratory) felt that as so much had been said about wind it might be worth while to comment on the tests which had been undertaken at the National Physical Laboratory. In the past engineers had been concerned with the question of aerodynamic instability only when designing chimney stacks, simply because these were the only structures with low enough periods of oscillation to be excited by the natural wind. It was a point to be noted that the Museum Tower was probably the first inhabited structure in Britain which was liable to this form of wind-induced oscillation. The fact that the oscillations in the model tests had been very small should not be taken as an indication that aerodynamic instability was unlikely to cause trouble on other tall buildings. Since the tests in question which had been carried out in 1961, the Laboratory had made further tests on building structures of some considerable height and had found that sections other than the circular had proved to be very much less stable. The most troublesome section would appear to be the square, a fact worth noting when considering buildings for which square and rectangular shapes were usually adopted.

247. It might be useful to state very briefly what information could be gained from model tests and what could not. The construction of the model was very briefly described in the Paper. Put simply, the design of such a model involved the scaling of the mass and stiffness distribution of the full-scale tower, as well, of course, as keeping a strict linear scaling of the external dimensions. What had not been scaled was the structural damping, a feature which had been mentioned by many previous speakers. Hence wind tunnel tests on the tower were carried out over a range of structural damping and it had then been necessary to make what could only be called a good guess for the probable value of structural damping of the full-scale tower.

248. This brought him to the question of full-scale measurement. While many instruments had been installed in the tower, from which no doubt a great deal would be learned, it was a pity that no instruments had been designed to measure the actual displacement of the top of the tower, to give some correlation between the model tests and the full-scale findings. One of the most important tests that could be carried out on the tower would be a measurement of the elusive factor of structural damping. If that could be done it would be a most valuable figure to have for future buildings.

249. The results of the laboratory tests had shown that the Museum Tower would almost certainly be completely free from any form of aerodynamic instability.

The following contributions were received in writing:

Mr W. J. Larnach (University of Bristol) wrote that he was particularly interested in the problem of what the Authors called the 'shaft design'. In Figs 20 and 21 the steady state wind loadings and the corresponding slope, bending moment and deflexion profiles were shown, but little indication was given of the way in which these were obtained. Were load tests made on the model, or were these results

obtained by calculation? It would in either case be informative to have more detail of the methods used and in particular of the degree of refinement if calculations were made. The exact solution for the bending of a free-standing cantilever of appreciable self weight and subjected to side loading was not easy. Even in the basic case of uniform cross section and uniform side loading the mathematics became involved, and for the Radio Tower it would seem that an exact solution in closed form could not be obtained; a numerical solution of the differential equation of bending would undoubtedly be required. The problem was made much more difficult by the presence of the yielding prop near the base.

251. The use of approximate methods could, however, be of some help in the case of the free-standing tower. The following comparisons would show that good estimates of deflexion could be made, but that estimates of bending moment were much less exact, and on the unsafe side.

252. If the uniform cantilever, external diameter D (Fig. 46) were considered and loaded by a uniform side loading of w /unit area to the right, using the energy method a solution could be obtained if a deflexion curve was assumed which fitted the boundary conditions. In this case it was convenient to take the form appropriate to a weightless cantilever, i.e. assume

$$y = a(x^4 - 4h^3x + 3h^4) \quad \dots \dots \dots (1)$$

in which a is a constant to be determined. The simple algebra of the energy method led directly to the value

$$a = \frac{wD}{3(8EI - qh^3)} \quad \dots \dots \dots (2)$$

where, in addition to the previously defined symbols,

I = second moment of area of strut cross section

q = weight of strut per unit length

E = modulus of elasticity of strut material.

253. It could be seen from equation (2) that there was a critical height h_c for which the cantilever became unstable and

$$h_c = \sqrt[3]{\frac{8EI}{q}} = \sqrt[3]{\frac{ED^2}{2d}} \quad \dots \dots \dots (3)$$

for a solid strut of material of density d .

254. The deflexion curve for a weightless cantilever could be shown to involve a value

$$a = \frac{wD}{24EI} \quad \dots \dots \dots (4)$$

If therefore a 'magnification factor' m defined as the ratios of the deflexions of the heavy and weightless cantilever, i.e. the ratios of the values of a in equations (2) and (4) was considered, it followed that

$$m = \left[\frac{1}{1 - \frac{qh^3}{8EI}} \right] = \left[\frac{1}{1 - \left(\frac{h}{h_c} \right)^3} \right] = \frac{1}{(1 - R^3)} \quad \dots \dots \dots (5)$$

where $R = (h/h_c)$.

For the massive strut combining (1) and (2), it is found that

$$y = \frac{wD}{3(8EI - qh^3)}(x^4 - 4h^3x + 3h^4) \quad \dots \dots \dots (6)$$

from which both slopes and bending moments might be derived.

FIG. 46: UNIFORM CANTILEVER

255. The *exact* solution of the same problem was not so straightforward. It was not difficult to show that the differential equation of flexure was

$$\frac{d^2 z}{dX^2} + KzX = 0 \quad (7)$$

in which $\left. \begin{aligned} z &= p - \Delta \\ p' &= dy/dx = \text{slope of the cantilever} \\ \Delta &= wD/q \\ K &= (h^3 q/EI) \end{aligned} \right\} \quad (8)$

and $X = x/h$

Equation (7) could be shown to have the form of Bessel's equation, and the solution was

$$z = X^{1/2} [AJ_{1/3}(\rho X^{3/2}) + BJ_{-1/3}(\rho X^{3/2})] \quad (9)$$

in which $\rho^2 = (4/9)K$. It would now be possible to integrate equation (9) to produce the deflexion curve given by

$$y = h\Delta \left(X - 1 - \frac{1}{J_{-1/3}(\rho)} \left\{ \int_0^X X^{1/2} J_{-1/3}(\rho X^{3/2}) dX - \left[\int_0^X X^{1/2} J_{-1/3}(\rho X^{3/2}) dX \right]_{X=1} \right\} \right) \quad (10)$$

The integrals of this equation were effectively carried out by integrating term by term the appropriate series based on

$$J_{-1/3}(a) = \frac{a^{-1/3}}{2^{-1/3} \Gamma(2/3)} \left[1 - \frac{3a^2}{2.4} + \frac{9a^4}{2.4.4.10} - \frac{27a^6}{2.4.6.4.10.16} + \dots \right] \quad . . . (11)$$

and when y was known slopes and bending moments could be evaluated. Numerical investigation had shown that usually fewer than six terms were required to evaluate the expressions in equation (10) correct to six places of decimals.

256. In Figs 47 and 48 the profiles of deflexion, y , and bending moment, M , were plotted in dimensionless form (for a variety of values R) along a homogeneous solid cantilever of circular cross section, using the exact solution for y (equation (10)), and evaluated using a computer programme.

257. As a comparison with the results obtained using the approximate energy method, a solid steel cantilever of 1 ft diameter was considered, although this was in no way meant to represent a realistic structure.

For steel take: $d = 490$ lb/cu. ft
 $E = 4.32 \times 10^6$ lb/sq. ft.

FIG. 47: REFLEXION CURVE FOR UNIFORM STRUT WITH UNIFORM SIDE LOAD

FIG. 48: BENDING MOMENT DIAGRAM FOR UNIFORM STRUT WITH UNIFORM SIDE LOAD

Then with $D = 1$ ft (equation (3))

$$h_o = 163.96 \text{ ft, say } 164 \text{ ft.}$$

(It was convenient to use equation (3) to calculate h_o , since it agreed almost exactly with the precise value.)

If the actual height was $h = 65.6$ ft then

$$R = \frac{h}{h_o} = 0.4.$$

If the effective wind pressure was $w = 10$ lb/sq. ft then from equation (8)

$$\Delta = \frac{wD}{q} = 0.025984.$$

Thus, $h\Delta = 1.712379$ ft.

Also $\frac{EI\Delta}{h} = 83\,997$ lb ft.

The results appeared in Table 5. Columns (1) and (2) were the calculated values of $(y/h\Delta)$ and $(Mh/EI\Delta)$ in the exact solution.

258. It would be seen that the exact and energy methods were in almost complete agreement in respect of deflexions, whilst the bending moments were considerably underestimated by the energy method. However, it would appear to be possible to improve the accuracy by taking y equal to the sum of a number of terms like that in equation (1). The weightless cantilever approach underestimated both deflexion and bending moments.

259. It was clear that in problems of this nature the self weight could have very appreciable effects on the bending moments, and should not be neglected without some investigation.

Mr K. Szmidt (Haynal and Myers) wrote that he had noticed that on Fig. 2 showing comparative outlines of various international buildings that the vertical scale was distorted above the 800 ft level. It affected adversely the proportions of the three tallest buildings (Empire State, Tokio, and Eiffel Tower) apparently reducing their slenderness. This did injustice to the designers and builders of these world-famous structures when comparing them with the three new radio towers (Dortmund, Berlin, and Museum). He presumed that this misleading comparison was unintentional and that it was a draughtsman's error.

Dr P. Selvanayagam (Civil Engineering Dept., Imperial College) wrote that in a tower of this height the consideration of aerodynamic stability for lateral oscillations was very important. Future towers were bound to be higher and aerodynamic stability would naturally be an important factor in the design. The danger of a forced resonance in the shaft and breeding of the amplitude could be minimized by introducing a damper in the structure.

262. It had been found by experiments that the vertical vibrations of a floor slab in a building near a railway track could be damped by introducing a visco-elastic material at the top of the slab and covering this with a thin layer (say $\frac{3}{4}$ in. to $1\frac{1}{2}$ in. thick) of concrete or screed as a restraining layer. In the case of a tower the lateral oscillations could be damped by using a visco-elastic material near the inner face of the shaft wall from the top to the bottom of the shaft (Fig. 49(a)). As there were cross slab floors it would not be possible from a practical point of view to introduce a continuous medium of visco-elastic material from the top to the bottom of the tower round the full circumference of the shaft, but a series of vertically continuous media of this material at regular intervals round the circumference of the wall (Fig. 49(b)) might be introduced without any reduction in the stiffness of the shaft or the slab floors. By choosing a suitable visco-elastic material whose properties were known it was possible to achieve a high damping factor for the composite structure.

TABLE 5: FREESTANDING CANTILEVER

X	Exact solution		Deflexions, ft			Bending moments, $lb. ft$		
	$(y/h\Delta)$	$(M_h/EI\Delta)$	Exact	Energy	Weightless	Exact	Energy	Weightless
0	0.06851	0	0.1173	0.1166	0.1092	0	0	0
0.1	0.05935	0.00485	0.1016	0.1011	0.0946	407.5	229.9	215.2
0.2	0.05023	0.01940	0.0860	0.0856	0.0801	1 629.5	919.5	860.7
0.3	0.04123	0.04362	0.0706	0.0703	0.0658	3 664.1	2 069.0	1 936.5
0.4	0.03248	0.07745	0.0556	0.0554	0.0519	6 505.7	3 678.2	3 442.7
0.5	0.02418	0.12077	0.0414	0.0413	0.0387	10 144.0	5 447.1	5 379.2
0.6	0.01658	0.17336	0.0284	0.0284	0.0266	14 562.0	7 746.1	7 746.0
0.7	0.00998	0.23494	0.0171	0.0171	0.0160	19 734.4	11 264.4	10 543.2
0.8	0.00474	0.30509	0.0081	0.0082	0.0076	25 626.5	14 712.7	13 770.8
0.9	0.00127	0.38326	0.0022	0.0022	0.0020	32 192.3	18 620.7	17 428.6
1.0	0	0.46879	0	0	0	39 373.5	22 988.5	21 516.8

FIG. 49: USE OF VISCO-ELASTIC MATERIAL FOR DAMPING LATERAL OSCILLATIONS
(a) CONTINUOUS LAYER; (b) IN STRIPS

263. In a cantilever beam of length L with a composite cross section as shown in Fig. 50 the natural frequency of vibration of the composite beam in its n th mode was given approximately by the expression

$$\frac{m\omega_n^2 L^4}{EI} = k_n^4 \left[1 - \mu d \left(\frac{k_n^2}{k_n^2 + \sigma} \right) \right]$$

where m = mass per unit length of beam,

EI = flexural rigidity of the beam,

$\omega_n = 2\pi f_n$ corresponding to the n th natural frequency of the beam,

h_n = frequency factor characterizing the n th mode which varies with the end conditions.

$$\mu = \frac{2d_2}{h_1}$$

$$\alpha = \frac{E_0 I_0}{EI}, \text{ where } E_0 I_0 = \frac{h_1}{2} [E_2 A_2 d_2 + E_3 A_3 d_3]_2$$

$$\sigma = \frac{2bGL^2}{E_2 A_2 h_1}$$

For a cantilever beam $k_1 = 1.875$, $k_2 = 4.694$, $k_3 = 7.855$, $k_4 = 10.996$, etc., satisfying the equation

$$\cos k_n \cdot \cos k_n + 1 = 0.$$

The above parameters were the same as for a homogeneous beam. It was possible to evaluate the EI value of the composite section of the tower shaft and apply the above equation.

264. It was rather difficult to estimate the damping factor with a layer as indicated in Fig. 49(b). However, one could predict from experiments carried out on other types of beam that the damping factor could be of the order of 0.1 to 0.3. Further research would have to be carried out on a concrete model to ascertain the exact value. The behaviour of the actual concrete structure would be different from the brass model that had been tested in the wind tunnel by the Authors. Concrete, because of its visco-elastic properties, had a better damping factor than brass or any other metal. In the case of a reinforced concrete beam high damping effect was produced by introducing the visco-elastic material about the neutral axis where the shear

FIG. 50: CANTILEVER BEAM WITH COMPOSITE CROSS SECTION

was maximum. In this case the stiffness of the beam was reduced. However, in the case of a circular shaft the stiffness of the shaft was not reduced by the introduction of the visco-elastic material.

265. There was no mathematical justification for the view expressed by the Authors that the exposed aerials had an important damping effect on the amplitude of oscillations. The introduction of visco-elastic material in the structure would certainly damp the amplitude of oscillations and there was every justification for its use in a high tower.

266. Another method of damping was to introduce horizontal dashpots below ground level, but the provision of these would be uneconomical when compared to the use of visco-elastic material in the shaft wall.

267. A $\frac{1}{4}$ -in. settlement of the foundation on one side relative to the other would cause a horizontal movement of about 3 in. at the top of the tower. It was desirable to make provision in the foundation for corrections to be made for the differential settlement. The provision of such a foundation would undoubtedly cost more, but this extra expenditure would be small when compared to the total cost of the structure and was justifiable in the long run.

268. He wished to thank Mr J. A. Agbasiere of Imperial College of Science and Technology for his valuable suggestions in the preparation of this discussion.

Mr M. Rees (Watermeyer, Legge, Piésold, and Uhlmann, Johannesburg) asked the Authors if they would say whether the natural frequency of the model tower was measured, and if so how it compared with the calculated full-scale value of 0.16 cusec. On the question of the results of the wind tunnel test, it would be of interest to have details of the scale factors (and their derivation) used to obtain the full-scale amplitude and frequency. Was the damping coefficient δ measured for the model, or could it be derived from the test results? What was the related full-scale value?

270. The curves of Fig. 12 showed the relation between half amplitude and wind speed and it would seem that, by extrapolation for the aerial sheeted-in section, the tower approached a state of instability under the (untested) wind velocity of some 50 mile/h. It would be of interest to know why the test was curtailed at a wind speed of 42 mile/h. The theoretical speed for instability was incorrectly given as 30 mile/h. It should surely be $5 \times 0.16 \times 50 = 40$ miles/h, so vindicating the order of the test results. Would the Authors also give the range of frequencies corresponding to the amplitudes of Fig. 12?

271. Did the Authors consider the effect of fatigue in the design of the tower shaft? Transverse oscillation under a high steady wind would result in reversible stresses in the transverse plane. From Fig. 12 and Table 4 an impression was given that the range of oscillation would be some 500–600 lb/sq. in. in the concrete and only 3000 lb/sq. in. in the steel. The alleviating condition, of course, was that the fibres most under stress in the longitudinal direction (those furthest to windward and to leeward) were removed from the fibres suffering the worst oscillating stress.

Mr R. W. Brims (retired, formerly Director, Brims & Co. Ltd) requested further information on the construction of the formwork. Why were two tiers of formwork used on the inside but three on the outside? Fig. 31 might have explained, but the photograph only showed that the central panel of the lowest tier appeared to have lost one of the vertical timbers. Was the shuttering designed by a specialist? How soon was the formwork struck?

273. The cantilever floors were exceptionally interesting. Fig. 35 was a little confusing until it was realized that this was in the B1–B7 range which was the conical part of the shaft. It would seem that the shutter frames were designed for about 30 re-uses, but that in this lower B1–B7 range they would project far beyond the concrete of the cantilever floor. The make-up gap at B3 was shown in Fig. 35 but in Fig. 34 there was no mention as to how the filling piece was supported.

274. Still relating to part 2 (construction) he referred to the diaphragm cut-off wall to the foundations constructed by the ICOS Company. Was the adhesion between steel and concrete quite certain under this method? He was somewhat alarmed at the possible unreliability of a reinforced concrete retaining wall 20 in. thick (nominal) placed in a trench 27 ft deep under liquid conditions not subject to visual observation. In § 107 the Authors mentioned that there was some weeping of water, stopped by pressure grouting. This probably occurred at the vertical stop-ends, but did the Authors make any trial cuts into the wall, after the dumping was removed, to test the quality of the concrete? Fig. 28 certainly showed a reasonable, if rough face.

275. He found two phrases used in the Paper especially attractive: namely that the operatives on the shutters climbed 'fly-like' on the working face and that the concrete mix was adjusted for adequate 'flowability'.

The Authors in reply expressed their appreciation of the many valuable contributions which had been made to the discussion.

277. Several speakers had referred to the choice of wind velocity and the sensitivity in response of the structure to peak gusts of short duration. These were the fundamental decisions to be made in the design of the tower and precise assessment was not at present possible on these questions. The accuracy of the assumptions could be checked in the future from the instrumentation which had been built into the structure and was intended to correlate the wind speed with the pressure and corresponding deformation of the tower.

278. The critical condition of stability was in respect of stiffness in order to satisfy the user requirement, and the primary stresses from wind forces were relatively low. During brief periods in which the design wind speed was exceeded, the additional deformation of the head might result in a loss of contact with the aerials of the adjacent stations and this limited risk had been accepted by the client as a reasonable balance of economy.

279. The 50-year return period was chosen as a practical limit which would cover the majority of the expected conditions within normal design limits and provide for abnormal circumstance within the safety margin. The increases in wind velocity for a return wind period of 100 and 200 years relative to the 50-year base were 7% and 13% respectively. The corresponding increases in pressure were 14% and 28% respectively. Because of the lack of precise information, the shape of the vertical

wind profile over cities was largely guesswork, but some readings had been taken since completion of the variation in wind speed over the height of the tower and these indicated close agreement with the wind profile which had been assumed in design as shown in Fig. 10. The wind profile was related to the datum recommended by the Meteorological Office¹⁴ namely 33 ft above 'effective ground level' which in large cities was specified as at 100 ft above the street level. It would greatly simplify comparison with other authorities if a common datum of 33 ft above ground level was adopted in all cases.

280. The choice of gust period was also indeterminate. Wind pressure was commonly calculated from velocity on the basis of the kinetic energy and this was satisfactory where the wind speed was constant, but the effective gust period must be of sufficient duration to build up a momentum in the air mass corresponding to the energy expended in deflecting the structure. The 15-sec gust was selected for the overall design of the tower from an approximate assessment of this type.

281. Meteorological instrumentation was not generally sensitive to the accurate measurement of mean wind speeds over short time intervals and records were normally quoted as peak values of uncertain duration. The maximum wind speed which had been recorded at the tower head since completion was 100 mile/h in the early spring of 1966. This however was for a short burst and corresponded to a 15-sec gust of approximately 80 mile/h. The measured rotation of the head was then 8.2 min of arc which corresponded to a deflexion of approximately 9 in. This might be somewhat less than the anticipated maximum loading but otherwise agreed generally with the expected performance.

282. For the particular conditions of this tower, the normal concept of load factor design seemed inappropriate. The critical section for elastic behaviour occurred in the region of the lower cantilever floors. At this level the primary conditions of combined wind and dead load only were low, but high secondary stresses were induced by the floors as indicated in Table 4. The bending stresses were, however, of opposite sign on each wall face and again reversed above and below the floor junction. On this account, under ultimate load conditions, it might be assumed that the secondary stresses would tend to cancel out.

283. No contributor had mentioned the potential relief in wind pressure which was induced by the 'down-draught' of the wind which followed the increasing horizontal velocity with increased height. Some preliminary work had been done with vertical velocity profiles in wind tunnels which had indicated that this effect might induce a reduction of 25% in the calculated horizontal wind load and appeared to justify a not-too-pessimistic appraisal of the many design factors involved. This down-draught commonly caused some discomfort to passers-by at street level in the neighbourhood of tall buildings with large exposed rectangular areas but it was not expected that the slender profile of the tower would set a sufficiently large volume of air in motion to cause marked effect at street level.

284. Considerable interest had also been expressed in the assessment of the aerodynamic stability of the tower. This was caused by 'vortex shedding' as explained in the Paper and for which the appropriate references had been given. There were two parts to this problem; firstly, the critical wind speed at which abnormal transverse movement occurred and secondly, the magnitude of the lateral deformation then produced.

285. The critical wind speed seemed to depend on the profile of the structure. A basis of calculation had been deduced for a simple cylindrical profile but, in practice, the critical velocity was beyond present methods of precise prediction. It would be seen from the test results shown in Fig. 12 that the Museum Tower, although of nearly cylindrical form, did not follow the expected pattern of transverse movement.

286. The second part of the problem of aerodynamic stability was to assess the magnitude of transverse deformation. The maximum amplitude of oscillation occurred when the energy input from the formation of the vortices was balanced by

the absorption of energy or 'structural damping' of the structure. This condition was measured in terms of the 'logarithmic decrement of structural damping' which was defined as the natural logarithm of the ratio of amplitude of successive cycles of oscillation from an initially deflected position in still air.

287. In most cases the amplitude of oscillation was defined in magnitude and linked to a particular speed. On this account it seemed appropriate to limit the use of the term 'aerodynamic instability' to those cases in which the amplitude of oscillation increased indefinitely at constant critical velocity. The model tests were made in steady winds and it was assumed that the wind gust would be of sufficient duration to induce the full amplitude of oscillation corresponding to that velocity. In the case of the tests on the model of the Museum Tower the amplitude of oscillation was defined in magnitude relative to the applied wind velocity at all speeds and by definition there was no tendency to 'instability'.

288. In this respect the present tests did not seem to confirm the advice given to the Chairman to avoid cylindrical chimneys with vertical striations. A smooth cylinder normally produced peak values of transverse oscillation at a relatively low critical velocity and large amplitude of oscillation. The Museum Tower was an 18-sided polygon in cross-sectional plan and the profile was not far removed from the 'cylindrical-striation' form. In this case, when the aerials were covered, the amplitude of transverse oscillation was high, and increased with increasing wind velocity, but appeared to be less than for a smooth cylinder at lower critical velocity.

289. Later tests on the tower at Birmingham,¹⁵ which was of notched square form in cross-sectional plan but otherwise of similar width and height, had indicated that this structure was sensitive to abnormal transverse movement at a critical velocity of about 90 mile/h. This appeared to agree with the '5ND' relation but the amplitude of oscillation was much less than that induced in the Museum Tower. Mr Whitbread's comments on the relative instability of square shapes did not appear to be confirmed by the comparative results on the two towers.

290. Some full-scale tests had indicated that for reinforced concrete chimney structures the value of the 'logarithmic decrement' varied between 0.05 and 0.10. The uncertainty as to the precise value of the 'logarithmic decrement' was overcome by testing the model through a full range of values. The maximum amplitude of transverse deformation was not oversensitive to variation in the decrement within this range. Further details were given in the quoted references.

291. The comparison between the oscillation trace of the tower when the aerials were enclosed and open appeared to indicate the value of the aerials in breaking up the vortices which induced abnormal transverse oscillation. If on some future occasion a different system of transmission was required, further consideration would be given, as necessary, to the means of containing the deformation of the structure within the required limits. In this respect also, the suggestions for introducing controlled damping were valuable.

292. The amplitudes of oscillation shown in Fig. 12 were based on a frequency of 0.16 c/s corresponding to a period of oscillation of 6 sec, and this was the calculated frequency of the full-scale structure. The model had been built to reproduce to scale the stiffness of the full-scale structure and the wind tunnel tests had confirmed the calculated frequency of the prototype.

293. Only a part of the oscillation trace for the 'sheeted-in' model was reproduced in Fig. 12. The wind tunnel tests had been taken up to a speed of 90 mile/h and there was no indication of instability within this range. The Authors did not agree with the evaluation of 40 mile/h for the critical speed of an equivalent cylinder. The '5ND' value for the 52-ft diameter cylinder was 30 mile/h. Such a condition was apparently representative of the major portion of the tower but a corresponding behaviour was not observed in the model tests.

294. The stresses due to oscillation were too low to be significant from the fatigue aspect. Torsional oscillation had been considered, but the departure from the

symmetrical form was slight and the prospect of torsional oscillation was therefore thought to be unlikely. An assessment of the dynamic magnification of the Stuttgart Tower had been made by Dr Leonhardt. He had concluded that this factor was not significant and no corresponding assessment had been made by the Authors on the present structure.

295. Several speakers had referred to the bridge link. The north-south joint on the tower side was arranged parallel to the adjacent main building for the full width of the bridge roof and floor slab. At the level of the bridge link the rotation of the deflected tower in the north-south direction and corresponding torsion in the joint was small. The relative settlement was likewise small and induced a non-repetitive set in the joint. A sliding collar involved immense difficulties of support and seemed unjustified within the anticipated range of movement.

296. The effect of the bridge link was to avoid unequal settlement and the action of the link restrained the tower from any tendency to tilt. There had been no practical prospect of increasing the size of the tower base. The hollow truncated base provided access for the complex arrangement of lifts and service communication from the basement of the adjacent building into the tower.

297. Some speakers appeared to suggest that wind effects could be ignored. This seemed at variance with the general trend of opinion but was an indication of the need to achieve a practical balance between caution and economy. The modified wind condition for which the structure was designed induced a wind load of 200 tons at an effective height of 350 ft. It was difficult to imagine that a force of such magnitude would be ineffective during the operative period of 15 sec.

298. The drag coefficient of 0.75 had been chosen for the 18-sided polygon after an examination of comparable wind tests. The 'aspect ratio' effect also produced a useful reduction in the drag coefficient.

299. The assessment of the elastic deflexion of the tower had been based on conventional area/moment methods with a variable moment of inertia. An 'exact analysis' had been made of the deflexion of the Museum Tower which allowed for secondary bending and this appeared to show an increase of less than 10% in the theoretical deflexion which was well within the accuracy of the wind load assumptions. The actual elastic deflexions of full-scale structures from 'live loads' had an obstinate habit of being less than the theoretical assessment by conventional methods. This was probably due to the inability of design to account fully for the inherent joint stiffness and variations in the elastic modulus, and this over-estimation appeared to have also occurred in the present case.

300. Shoot lines from the tower to adjacent stations on the outer fringe of London had been arranged as far as possible so as to avoid interference from tall buildings which might be constructed in the future, but there was scope for repositioning the aerials in both the horizontal and vertical planes should this be necessary.

301. Alternative design exercises were made into the possibility of supporting the floors at their outer periphery by columns or hangers carried from a stiffened cantilever tower floor or upper roof. Both these solutions would have made for lighter floors and overall economy but were rejected in order to avoid obstruction to the aerials and the glazing. These subsequently proved to be unnecessary precautions.

302. For electronic reasons the reinforcement and other metal components of the tower were required to be electrically bonded to form an earthed 'Faraday cage'. This of course constituted a 'leak path to earth' for the lightning which had to be accepted and made the provision of the conventional small copper tapes redundant. From a review of the available evidence it appeared that a lightning strike was not likely to damage the reinforcement.

303. On the basis of relating the momentum of the air mass to the energy of deformation, the cladding components ought to be designed for a shorter gust period than was appropriate for the structure as a whole. For the glazing panels a period

of 3 sec was adopted corresponding to a wind speed and pressure of 120 mile/h and 37 lb/sq. ft respectively.

304. In the design of the prestressed base slab a coefficient of friction of 0.3 had been assumed between the slab and the oversite concrete which were separated by an oiled membrane. Strain gauge tests during stressing indicated that the resistance to movement was less than had been anticipated.

305. Tests were made to check the effect on the bond between the steel and concrete of the bentonite solution which was used to stabilize the excavation of the diaphragm wall. These tests indicated that the loss of bond was negligible at working stress, at least in so far as high tensile deformed bars were concerned.

306. The lifts were designed to move at a maximum speed of 1000 ft/min and were the fastest known in Europe. It was only worth using a lift of very high speed for a long run, as in the present instance.

307. A more sensitive instrument has since been installed to measure the change of slope at the head of the tower. This was based on twin pendulums at right angles, related to a gyroscope datum and sensitive to a response within a period of approximately 0.05 sec.

308. The very intense examination of the design which had been invoked had, during the discussion, perhaps tended to overlook the ingenuity of the methods employed for the construction of the tower and the Authors were most grateful to Mr Prior for his valuable commentary.

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