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“The Reconstruction of Ten 305-Foot Tubular Steel Radio-Masts in Reinforced Concrete.”†

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Mr. Kurt Billig observed that as, according to the Author, the original mast sections would be allowed to rust away after the reconstruction, it would be necessary to ensure the stability under maximum load without the presence of the existing mast sections. The description of the mast showed that the horizontal and vertical flanges of the old tubular steel sections were embedded in the new reinforced-concrete structure, and that over more than 200 feet of height there was only a $1\frac{1}{2}$ -inch thickness of concrete between the outer edge of the circumferential flanges and the outer face of the reconstructed masts. Could the extent of the rusting be foretold? If the flanges did in fact rust, would they not expand and produce circumferential cracks in the mast, thereby allowing progressive deterioration to take place?

Mr. Billig believed that tower structures, chimneys, silos, pylons, etc., presented an ideal field for the application of pre-stressed concrete, which hitherto had been completely neglected. That statement could be illustrated by a comparative design of a similar 305-foot tubular radio mast in pre-stressed reinforced concrete (P.R.C.), the main numerical data were given in Table I.

The proposed P.R.C. mast had an outer diameter of 4 feet 3 inches at the base and a taper of 6 inches in 305 feet on each side. The thickness of the concrete shell was 3 inches at the top, increasing to $4\frac{1}{2}$ inches at the base. It was compressed by twin-twisted high-quality steel wires of standard wire gauge 11, sixty at the base, decreasing to twenty-eight at the top section.

To construct the P.R.C. mast an inner mould of steel sheeting was fixed against the tubular steel by means of spacer-blocks. The twin-twisted compressor wires were anchored in the existing foundations and positioned

† J. Instn Civ. Engrs, vol. 27 (1946-47), p. 113 (Dec. 1946).

TABLE I.
DESIGN OF 305-FOOT RADIO MAST IN P.R.C.

Position.	Mast section.	Level above ground : feet.		Concrete shell.			Special grade compressor wires.			Moment of inertia of P.R.C. cross section.	Dead weight of P.R.C. mast.	Approx. magnitude of bending moment.	Concrete stresses.			Position.
		outer diameter.	thick-ness.	c/5-al. area.	No. of wires.	c/5-al. area.	ratio.	compr.	due to dead and live loads.				effective preliminary.	total compressive.		
		ft. in.	in.	sq. in.	—	sq. in.	per cent.	in. ⁴	lb.	lb.-ft.	44	44	294	338	250	E
		305	3	3-00	3-00	339	0-592	54,900	—	10,300	44	44	294	338	250	E
		275	3	3-18	3-15	367	0-592	63,300	11,500	47,800	214	150	271	485	121	D
		245	3	3-36	3-30	395	0-592	72,500	23,000	49,100	226	110	251	477	141	D
		207	3	3-85	3-48	431	0-762	84,000	39,800	53,200	255	71	296	551	225	C
		170	3	3-66	3-66	467	0-931	96,200	56,700	56,000	276	34	336	612	302	C
		130	3	3-90	3-86	510	0-183	114,000	78,000	62,800	305	—	306	611	307	B
		90	3	4-06	4-06	553	0-199	132,000	99,200	72,000	335	—	334	669	359	B
		45	4	4-23	4-28	605	0-210	155,000	127,600	78,500	361	—	352	713	413	B
		0	4	3-00	4-50	656	0-194	178,000	156,000	83,800	382	—	325	707	419	A

NOTE : (1) Effective pre-tension in steel, 75 tons per square inch.
 (2) If provision is made for the P.R.C. structure to carry the old steel mast, the pre-compression of the concrete may be reduced.
 (3) There is no tension in the concrete of the P.R.C. mast.

against the horizontal flanges of the old structure, but the internal diameter of the P.R.C. mast was everywhere slightly wider than the flanges; the new and old structures remained separated by a small gap. The mast was concreted in, say, five sections of 60 feet each. At the level of 60 feet above ground a frame was secured against the steel structure and each of the vertical wires was stretched by a small hydraulic jack to a force of $1\frac{1}{2}$ ton, and clamped in a stretched condition. Then the outer mould was placed and the concrete was cast in the forms.

When the concrete had attained a strength of 2,500 lb. per square inch, the wires were released from the frame and compressed the annular concrete mast evenly over its whole cross section. The same process was repeated over the next section of 60 feet of height, with the only difference that fifty-two wire units were stretched at the level 120 feet above ground whilst eight stopped short at the 60-foot level. The number of compressor wires decreased gradually with the height of the structure and in the topmost section was reduced to twenty-eight, whilst the concrete cross-section was only 52 per cent. of the sectional area at the base.

The simplicity of the design of such a mast in P.R.C. would be evidenced by the design of mast section A at the base:—

Outer diameter, 4 feet 3 inches; inner diameter, 3 feet 6 inches; thickness of wall, $4\frac{1}{2}$ inches.

Cross-sectional area of concrete $A_c = \pi \times (25.5^2 - 21^2) = 655$ square inches.

$$\text{Moment of inertia } I_c = \frac{\pi}{64} (51^4 - 42^4) = 178,000 \text{ in.}^4$$

Dead weight of P.R.C. mast, 160,000 lb. = 71 tons.

Estimated bending moment (see Paper):

$$M = 78,900 \times \frac{51}{48} = 83,800 \text{ lb. ft.}$$

Maximum stress in the concrete, if the section were homogeneous:

$$c = \frac{160,000}{655} \pm \frac{83,800 \times 12}{178,000} \times 25.5$$

$$= 244 \pm 144 = \begin{cases} 388 \text{ lb. per square inch.} \\ 100 \text{ " " " } \end{cases}$$

To ensure that the concrete remained always compressed, the designer should choose a value of pre-compression that would safely exclude tension under the most unfavourable loading condition that might occur. In determinate structures the tension due to external load might be ascertained relatively easily and the amount of pre-compression chosen accordingly. In an indeterminate structure, such as the stayed mast under consideration, the discretion of the designer in the choice of the preliminary compression was much wider, especially if the statical calculation indicated only small or no resultant tensile stresses in the concrete.

In the case under discussion it would be assumed that a pre-compression of, say, 300 lb. per square inch represented a satisfactory reserve for any possible tension which might be caused by external loads.

The total pre-compression was $655 \times 300 = 196,500$ lb. = 88 tons.

Permissible stress of special-grade steel (see below) = 75 tons per square inch.

Required cross-sectional area of steel $A_s = 88$ tons : 75 tons per square inch = 1.17 square inch.

1 wire of S.W.G. 11 = 0.021 square inch ; diameter = 0.116 inch = 3.0 millimetres.

60 twin-twisted wires of S.W.G. 11 = 1.26 square inch, spaced at $2\frac{1}{2}$ -inch centres at ground-level.

The reconstruction of the mast in P.R.C. entailed a maximum working load on the foundation of about 130 tons, in comparison with about 170 tons in the actual reconstruction. The load of the original steel structure being 60 tons, the additional weight due to the reconstruction in ordinary reinforced concrete was 110 tons, whilst according to Mr. Billig's proposal it would be 71 tons. Thus a P.R.C. mast would require less than two-thirds the quantity of concrete and its foundation would be considerably lighter. The minimum compressive strength (works cube test) specified for the reconstruction was 3,500 lb. per square inch at 7 days ; and no better quality would be required for Mr. Billig's present proposal, especially if account were taken of the improvement in strength of a concrete which matured under a constant compression such as that introduced by the released wires.

The economy in steel was still more evident, amounting to a saving of about 90 per cent. in weight. The steel required for that type of P.R.C. structure was of 140/150 tons tensile strength and a conventional yield limit of 120–130 tons per square inch. Such steel could not be usefully employed in ordinary concrete construction. Although that was comparatively expensive—about £75 per ton—a substantial saving could still be derived from its use.

The calculation of stresses in a P.R.C. mast from given loadings was easier and more certain than for ordinary reinforced-concrete masts, where the moment of inertia of a given section was a function of the magnitude and position of the axial thrust, the bending moment, and the tensile strength of the concrete. On the other hand, the moment of inertia of a given P.R.C. section was a function only of its geometrical shape and of nothing else. Owing to the permanent compression of the material, the inherent tensile strength did not enter into the design.

By the pre-stressing process part of the compressive strength of the concrete was transferred to the tensile side, and within that artificially created tensile strength the concrete was completely elastic. Any deformation of the mast within those limits was wholly reversible. That was of especial importance in a structure like the stayed mast, where the effects of

wind, dead load, aerial load, the reactions of the stays, and many other factors caused a continuous change in the stresses of the structure. The resilience of the pre-compressed mast should be an outstanding advantage over the ordinary construction.

Mr. S. K. Ghaswalla observed that the Paper formed a valuable contribution to the literature on the design and construction of long reinforced-concrete masts and, so far as he was aware, was probably the first of its kind on that subject. The problems of wind-pressure estimation and analysis, as well as the structural design of masts under those varying loads, covered such diverse fields as physics, aerodynamics, and meteorology, and thus presented a challenging and complex façade to the designing engineer. Wind-tunnel analysis,¹ used nowadays for the determination of the exact aerodynamic properties of complex-shaped bodies, would, it appeared, throw considerable light on the present problem of pressure-distribution around a horizontal section of a cylindrical object. Such an investigation, if systematically carried out, would also be of considerable use in the design and analysis of tall chimneys and high water-towers.

Moreover, the Author had not overlooked the important question of temperature-stresses. In fact, it was an indication of awakening to important but neglected facts that thermal action in structures had formed the subject of discussion in recent technical papers. It was a pity, however, that the Author had not given detailed calculations for the evaluation of those temperature-stresses, presumably because he considered that they would assume the same form as the basic equations in chimney design. Mr. Ghaswalla could not entirely agree with that idea, because the structure described was fundamentally different from an ordinary chimney, in that it consisted of a steel tubular section of very great height, surrounded by a reinforced-concrete shell. In view of the fact that there were appreciable volumes of two different materials (steel and concrete) and that the thermal changes were more pronounced on the outer surface than in the inner core, it would have been preferable if the Author had analysed the effects of temperature-variations due to both solar radiation and fire. A detailed survey on that subject had been carried out by Carl C. H. Tommerup² and by Roy W. Carlson³ in the United States. Ever since J. B. Fourier⁴ had first raised the question of flow and absorption of heat in solid bodies in his classic memoirs, the subject had been treated extensively by physicists and mathematicians of international repute like Laplace, Poisson, Lord Kelvin, and Riemann, and more recently by engineers like Gurney, Heilman, King, McAdams, Newman, Preston, Schack, and Schmidt.

Mr. Ghaswalla hoped that the problems of thermal insulation would

¹ S. K. Ghaswalla, "Wind Tunnels," *Science & Culture*, Dec. 1943, p. 224.

² Carl Tommerup. *Proc. Am. Soc. C.E.*, June 46, p. 759.

³ Roy. W. Carlson. *Journal, Am. Conc. Inst.*, Nov.-Dec. 1937, p. 89.

⁴ "Théorie du Mouvement de la Chaleur dans les Corps solides." *Acad. Française*, 1812.

be more widely studied by engineers and that their important implications would be realized in practice.

Mr. Herbert Tooley observed that the Author had devised an elaborate and complicated set of equations in Appendices II and III to determine the relationship between stay tensions, movements of mast, etc., but upon examination it would be found they all were based upon the values of v , the elastic stretch of the stays, and the equation

$$S = L + \frac{v^2 L^3}{24T^2}$$

Adapting that equation to $L - S$ (in inches) = $\frac{v^2 L^3}{2T^2}$ and calculating the stretch of the stays for one tension only, which might be an easy divisor of E , Mr. Tooley, using the method described by him in 1941,¹ had devised a system of graphs which, as they depended upon the same essentials as the Author's complex equations, should be equally correct but only required the above-mentioned simple equation to be used.

The graphs had the advantage that if, after preliminary calculations, it seemed advisable, say, to alter the initial tensions, without further calculations, by means of an appropriate scale drawn on tracing paper laid over the graphs, the alterations in stay tensions and movement of mast could be read off, whilst similarly, for an alteration in movement of mast, alterations caused thereby to stay tensions including the initial tensions could be read off; surely that was better than having to solve again the Author's complex equations.

Mr. Tooley, with his limited knowledge of mathematics, was chary of raising other points; but why start with the assumption that the maximum aerial pull acted in the same direction as the wind, which it rarely did? The Author had directed attention to the importance of the changes in stresses caused by the aerial due to changes in direction of the wind. Mr. Tooley had found it easier to deal with the matter by making the preliminary design without (but not forgetting) the aerial, and then considering the aerial pull by imagining the direction of the wind as constant and the aerial pull as boxing the compass and varying in amount (of course, the reverse as to direction was the actual case); the effect of the aerial pull could then be studied. That seemed to lead to no more work and it was better than starting with a false assumption as to the aerial.

If initial tensions were calculated with the wind in the plane of the windward stay, so that the mast was deflected to a predetermined line, and the deflexion was then recalculated with the wind in the opposite direction in the case of a three-stayed mast, and at 45 degrees in the case of a four-stayed mast, it would be found that the deflexions altered irregularly, causing the mast to assume a crooked line, and adjustments would

¹ "Notes on Determining the Stresses in the Guys of Guy-Supported Masts." J. Instn Civ. Engrs, vol. 15 (1940-41), p. 220 (Jan. 1941).

have to be made to reduce the discrepancies to the minimum ; would not that render the calculations of reactions by the three-moment theorem inaccurate ?

In Appendix III, t_1 was stated to be at its maximum for three-stay masts when $\phi = 30$ degrees ; but because of the greater windage on the lee stays, t_2 at $\phi = 0$ degree was so much greater than t_2 when $\phi = 30$ degrees that t_1 was nearly always at its maximum when $\phi = 0$ degree.

In the comparison of maximum deflexions the value of h depended upon so many factors that equation (23) and the value of $2h$ given for three-stay masts were often very far from correct ; confirmation of that statement would be found in the Tables accompanying Mr. Tooley's Paper referred to above, which were not published but could be seen in the Institution Library.

The Author, in reply, observed that the risk of extensive corrosion of the embedded flanges of the original mast was not regarded as serious, but admitted it could not be entirely excluded. Such corrosion could, however, take place only "end on," and in comparison with that which occurred in the 30 years' life of the original sections, could be expected to be negligible during the anticipated life of the new mast. The external paintwork of the mast was in good condition before reconstruction and the prospect of corrosion of the flanges by the creeping of moisture along their faces would be limited and would not mature until the original circular sections completely corroded away. The retention of access to the interior of the masts would permit examination and would allow timely corrective measures to be taken if necessary. To have cut away all flanges or to have specified such an increased annular thickness of concrete as could have borne the required load itself, and have allowed for the necessary reinforcement, would have entailed greatly increased expense and was deemed unjustified.

Much could be said for Mr. Billig's suggested design in pre-stressed reinforced concrete giving, as it did, certain important advantages which he had clearly expounded and with which the Author was in complete accord. Other than on the question of ultimate strength, the Author's criticisms were mainly confined to the difficulties of its application to the present problem. One feature of pre-stressed construction was that although, in such designs, substantial economies in steel, and to a smaller extent in concrete, could be shown on the basis of working stresses, the ultimate loads for the structure might not be so high as for the corresponding unpre-stressed construction. On p. 129 the Author had discussed the necessity for the structure to withstand without failure some agreed multiple of its working load, and he believed that multiple would be less for the pre-stressed construction than for the original design. He considered that when, as in the present case, stresses were not directly proportional to load, a better criterion of safety was given by that multiple (see p. 139) than by the working and failing stresses of the materials.

In the detailed proposals for the present design the major practical difficulty foreseen was that of pre-tensioning the wires. As stated on p. 145, the greatest compressive load which it was felt prudent to allow on an existing mast, in view of the known unsound condition of the latter, was limited to a few tons only. The pre-tensioning of sixty wires at $1\frac{1}{2}$ ton each, added to the weight of the tensioning-frame, jacks, and working cage, would produce a load exceeding 100 tons—an entirely prohibitive load for the existing structures. (It might be interesting to consider the design of shuttering able to accept that load—that should not be impossible if too long a pre-tensioned section were avoided, and the method would be applicable for chimney and similar construction where no central structure first existed.)

The use of such thin annular sections as 3–3 $\frac{1}{2}$ inches over the upper half of the mast was not favoured, because they were difficult to provide accurately and a serious loss of strength would result from imperfections in their construction. Moreover, with very thin sections, a danger of failure would result from the very high negative pressures discussed on p. 120, against which, and for temperature stresses, the mast proposed by Mr. Billig did not appear to be reinforced.

Experience in the manipulation of the external steel shutters had led the Author to suggest that the internal steel shutters proposed by Mr. Billig would not be recoverable, particularly as a 60-foot length of mast was to be concreted at one time. Although the pre-stressed design employed less steel for reinforcement than did the original design, the total quantity of steel in the reinforcement, tensioning-frames, and non-recoverable shutters would exceed that of the original design. Admittedly, however, the use of internal shutters was not an essential feature of the pre-stressed construction, and doubtless it had been advocated by Mr. Billig to obviate the risk to which he had drawn attention in his opening remarks.

Finally, as discussed on pp. 131 and 132, the employment of many fine wires in the pre-stressed design would have permitted no safeguard against local failure of the concrete compared with the use of the heavy rods of the adopted design to which that construction was peculiarly suited.

In the calculation of stresses due to changes of temperature, mentioned by Mr. Ghaswalla, allowance was made for the presence of the original steel sections although the good thermal conductivity of the latter and its intimate contact with the reinforcement did much to equalize the temperature differences which might otherwise occur. The stresses from that cause were found to be comparatively low and accuracy in their assessment was unimportant. Additionally, since maximum bending stresses in the mast occurred under storm conditions (a wind of 100 miles per hour) it could be assumed that extreme differences in temperature due to solar radiation, as distinct from changes in the ambient temperature, would not occur simultaneously.

No material of a combustible nature had been used in the construction of the masts, nor in their vicinity, and the contingency of fire could be reasonably excluded. The accidental local heating of a mast at its head by, for example, extensive arcking at an aerial triatic termination might conceivably cause damage at that point, but could not endanger the stability of the whole.

The method of initial design advocated by the Author, in which from the known reactions at a stay level the necessary initial tension in stays could be determined explicitly from equations (15) or (26) of Appendix III, avoided the need for the graphical treatment advanced by Mr. Tooley. Doubtless the use of tension-deflexion graphs would facilitate the choice of alternative initial tensions in stays to suit other loading conditions if the reactions and deflexions at the different stay-levels were not independent. But that independence, in which a variation of the initial tensions of stays at one level altered the deflexion and stresses in the mast under load at all other levels, unfortunately required the simultaneous evaluation of reactions and deflexions on the basis of the data given in *Fig. 4*. The Author had found that to do that involved excessive trial and error and had therefore resorted to the method given in Appendix V, which, if approximate, was at least explicit.

The assumption of maximum aerial load in the direction of the wind had been instanced to illustrate a typical calculation, although the Author had stated (p. 136) that that condition was not necessarily the most severe. In that respect Mr. Tooley's method appeared to imply that the direction of the aerial pull was constant in relation to the line of the stays, irrespective of the direction of the wind. That might be nearly true for some aerial systems but, as stated on p. 139, the aerial load at Leafield might vary in direction through nearly a right-angle, depending upon the direction of the wind.

The theorem of three moments had not been used, nor had it been mentioned by the Author. It was deemed unsuited to the problem although, in fact, it could, in its more advanced form, take account of varying deflexions. Mr. Tooley's earlier comments had been answered in the reference made above to Appendix V.

The maximum value of t , given in Appendix III, was based, as stated, upon the assumption of a negligible value of t_2 . Although, in all designs examined by the Author, that was found to be so, he agreed that that approximation might sometimes be unjustified.

The relative deflexions for three- or four-stay structures (at any one level) given in equation (23) were theoretical maxima and, as such, demonstrated the increased deflexions to be expected when using three stays instead of four per level, although deflexions might not, in practice, always attain those maximum values.