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IIIa

The Design and Testing of a Cable Beam Structure for Prefabrication

Projet et essai d'une structure en câbles pour la préfabrication

Entwurf und Versuch an einem Kabeltragwerk für die Vorfabrikation

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1. Introduction

The objective of this work is to show that it is possible to design a cable beam structure that compares favourably in cost, at the lower range of long span roofing, with conventional methods of construction. The design of the boundary structure has been considered in this context and particular attention is paid to the method of construction in order to ensure that site work and the erection of the cable beams is undemanding of the contractor's skill and ability.

The experimental work gains greatly from the fact that a full scale model is being used and it is intended to use this advantage to establish the effect of construction errors and dimensional variation on the behaviour of the structure.

The priorities are considered generally as follows:

(a) The boundary structure should comprise simple frame elements of weight and dimension convenient for transportation and handling and should have inherent stability whilst final adjustment and alignment is being made.

(b) The cable beams should be easy to assemble and lift and with simple joint details not requiring close tolerances on dimensions.

(c) The final alignment of the cable beams should be by means of the forces in the cables and not dependent upon the accuracy of alignment of the boundary structure.

(d) The steel tendons and anchoring devices of the beams and the required jacking equipment should be readily available as for example such as used in prestressing.

2. Description of Cable Beam Structure

Each cable beam takes the form of two pairs of 0.5 in (12.8mm) Dyform plastic covered prestressing strands in reverse catenary, held in position by vertical 1.0 x 0.25 in (25.4 x 6.35mm) steel ties. A pair of cable beams of 75 ft (22.8m) span and 9 ft (2.74m) apart form the unit under test. The strands are anchored into the concrete end frames with standard prestressing grips at heights of approximately 14 ft (4.3m) and 21 ft (6.4m). The end frames are prestressed through slots in their vertical columns to the floor of the laboratory, thus each pair of end frames together with a portal unit produces a stable structure for the erection of the cable beams. The cable beams were assembled on the ground and were easily lifted into position. Final alignment was achieved by adjusting the tension in the cables. Fig. 1 shows the structure before the roof cladding, troughed aluminium sheeting, was fixed in position.

3. Experimental Work

The roof cladding for cable truss structures of the type investigated is normally designed using either troughed steel or aluminium sheeting spanning between the trusses and attached either to the suspension cable (SC) or prestressing cable (PC), depending upon the architectural requirements. Thus purlins are not used and both the weight of the cladding as well as snow and wind loading is applied transversely along the cables and not only at the joints as assumed in the usual analysis of cable structures.

In carrying out the experimental work on an individual truss this was taken into account by applying the load in the form of 50-lb sandbags at eight load points for each loaded cable link as well as at the joints with increasing loads of $q = 50$, 100 and 150 lbf (222, 445, 667N) per load point, the maximum load corresponding to 20.28 lbf/ft^2 (971 N/m^2). The following six loading tests were carried out:-

(a) increasing load on full span (LOFS) and

(b) increasing load on half span (LOHS)

both on: (i) the suspension cable

(ii) the prestressing cable and

(iii) the whole structure with cladding supported on the suspension cables.

When testing the individual truss, the sandbags were supported from the load points with nylon ropes. When the whole structure was tested the sandbags were placed on top of the cladding. The positions of loading on full and half span and a diagram of the truss are shown in Fig. 2a.

The experimental and theoretical deflections for a loading of 150 lbf (667N) per load point are shown in Fig. 2b and 2c. Fig. 2d shows the differences in calculated deflections using the theories described in the next section. The experimental and theoretical curves for the forces in the end links of the top and bottom cable are shown in Fig. 3a and 3b.

To determine the natural frequencies and the damping of one truss, the truss was pulled down at the centre in turn by a force of 25, 50, 100, 150 and 200 lbf (111, 222, 445, 667, 890N) which was then released.

The vibration of the truss was transmitted by a displacement transducer to a storage oscilloscope yielding a lower natural frequency of 9.4 cycles per second. This was repeated for the whole structure with the cladding in position yielding a natural frequency of 3.95 cycles per second.

To gain some idea of the damping of the trusses with and without cladding the above was repeated and the time for the vibrations to die down measured. The results of these tests are shown in Fig. 4 and the vibration for release loads of 150 lbf (667N) per truss without and with cladding are shown in Fig. 5 and Fig. 6, respectively.

4. Theory

The analysis of cable beams with the load applied along the cables are like the cases when the load is assumed supported at the joints based on minimisation of the total potential energy W by an iterative process. In the latter case, when the tension in the cables are sufficient to ensure linear elastic behaviour of the cable links, this implies descent on an energy surface which is quartic in the displacements. The minimising steps are usually taken in either the Newton-Raphson, Conjugate Gradients or Steepest Descent directions, a distance S to a point where W is a minimum in that direction (Ref. 1. and 2.). Thus it can be proved that at the (k+1) iterate,

$$W_{k+1} = C_1 S_k^4 + C_2 S_k^3 + C_3 S_k^2 + C_4 S_k + C_5 \tag{1}$$

where C_1 to C_5 are functions of the displacements at the end of the kth iterate to powers varying from one to four.

Thus S for each iteration can be found from

$$\frac{W}{S_k} = 4C_1 S_k^3 + 3C_2 S_k^2 + 2C_3 S_k + C_4 = 0 \tag{2}$$

When the load is applied along the cables the cable elements cannot any longer be regarded as components with linear load/extension characteristics. When this is taken into account together with the change in the potential of the loading due to the sag of each cable link, it can be shown that the descent takes place upon an energy surface which is octal in the displacements. In Ref. 3 it is shown that equation (1) above becomes

$$W_{k+1} = C_1 S_k^8 + C_2 S_1^7 + C_3 S_k^6 + C_4 S_k^5 + C_5 S_k^4 + C_6 S_k^3 + C_7 S_k^2 + C_8 S_k + C_9 \tag{3}$$

when C_1 to C_9 are functions of the displacements at the end of the k^{th} iterate to powers varying from one to eight.

Using the method of Conjugate Gradients the above theory has been used for the analysis of small cable beam models for which the differences between theoretical and experimental results varied from 5% to 8% when the loading was placed on the prestressing cables. One part of the current work is to see if this theory also will yield similar differences when applied to a full-scale structure.

5. Conclusions

After the stressing of the cable beams some movement took place of the tie fastenings over the plastic sheeting. This reduced the prestress in the top and bottom chord from 16,000 to 11,000 lbf. During the stressing no movements of the end frames were evident.

The load tests showed that load applied to the prestressing cables results in larger displacements and cable forces than when applied to the suspension cables. The stiffening effect of the aluminium sheeting was as expected, negligible.

The difference between theoretical and calculated results was greatest for loading on half the span only. This is thought to be due to small relative movements of the cables at the central joints. When the load is supported on the top cable the two theories give very nearly identical results but when the load is supported on the bottom cable the simplified theory underestimates both forces and deflections. The loss of prestress caused the prestressing cables to go slack when the last load increment was applied. The load intensity at which this occurs was accurately predicted by the analysis. The damping effect of the aluminium sheeting can be seen in Fig. 4. It has a negative rather than the expected positive effect, which must be due to the fact that the sheeting acts as a spring in which energy is stored. The tests described should be considered as a preliminary investigation which will be continued when the tie joints have been modified, a fact which has been most usefully made evident by the use of a full-scale model.

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Summary

This paper contains the description and testing of a cable beam structure which has been developed for prefabrication and low cost. The experimental results have been compared with the results from two different non-linear theories indicating for which cases the two different theories are applicable.

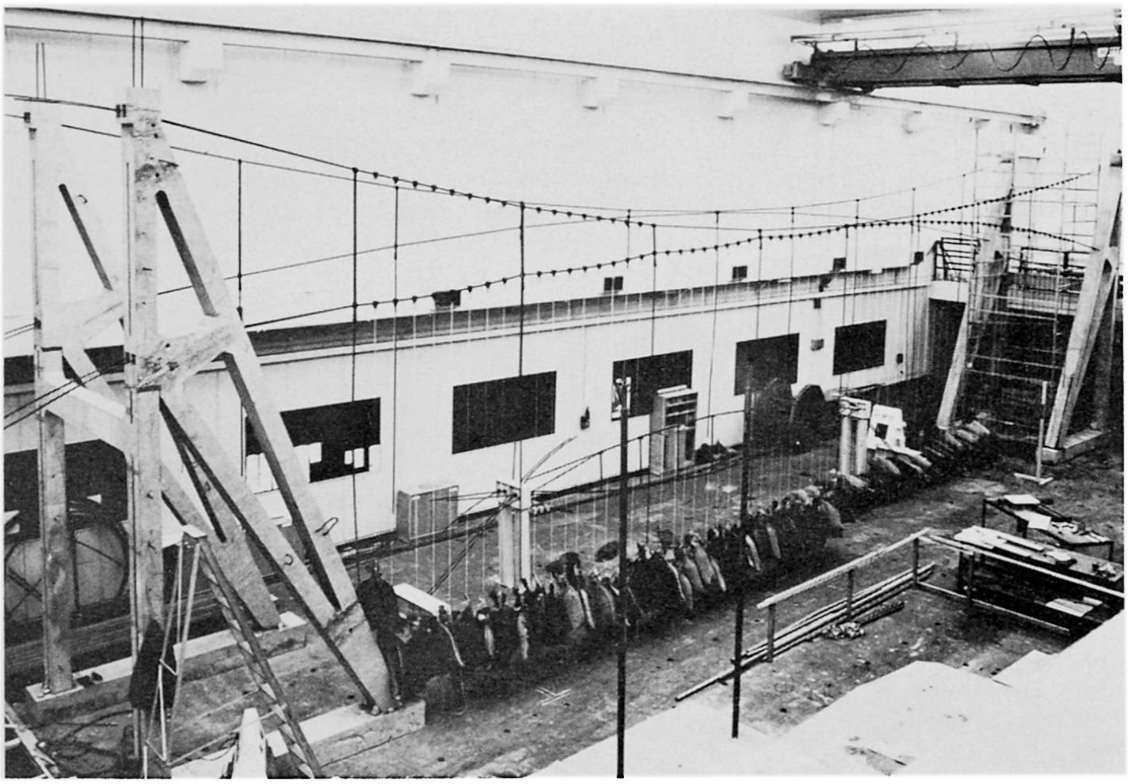


Fig. 1.

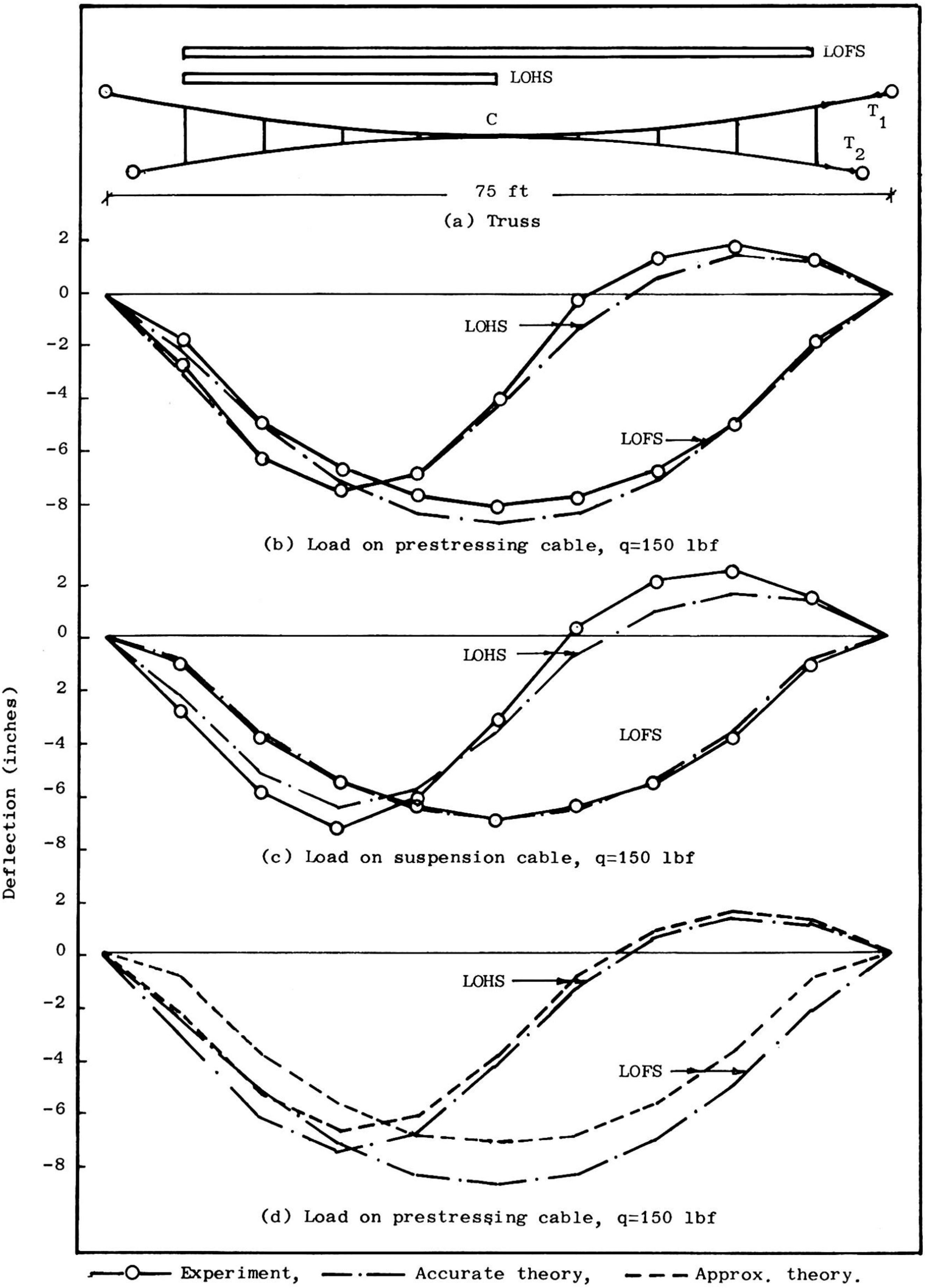
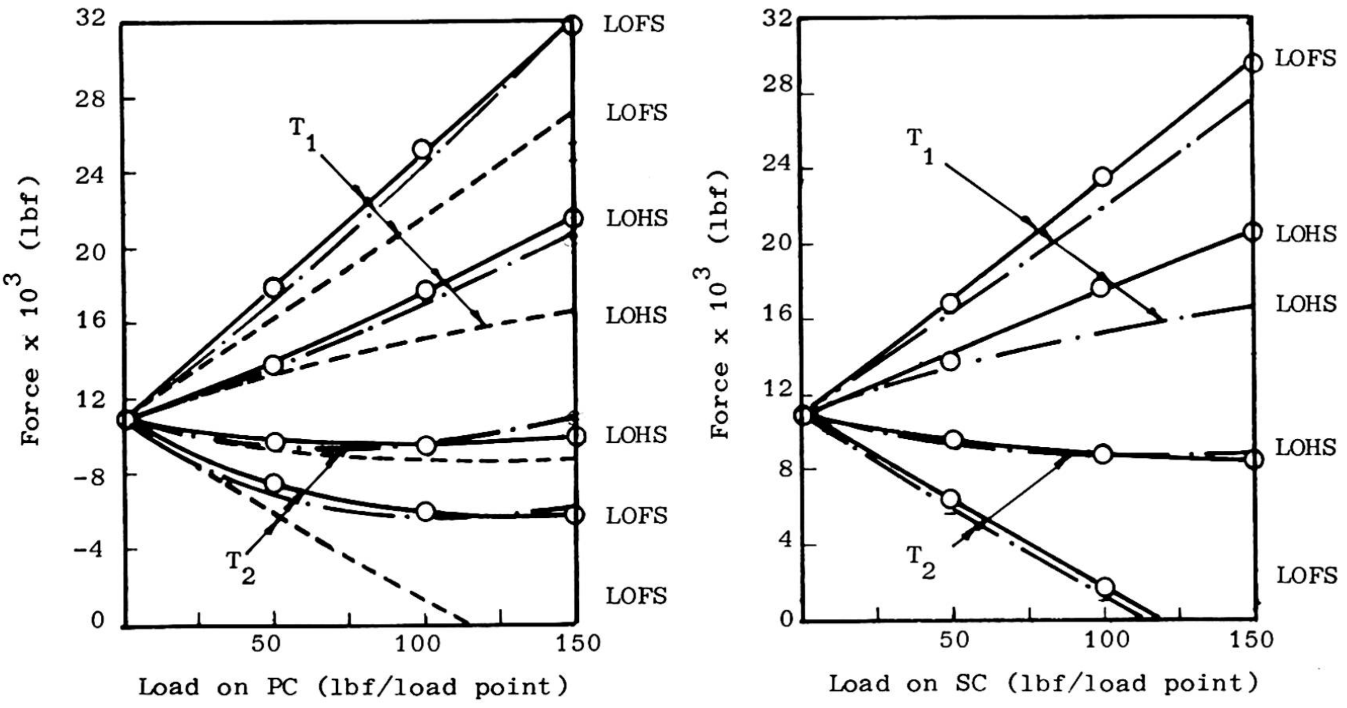


Fig. 2. Deflection diagrams of cable truss



(a) (b)

Fig. 3. —O— Experiment, —·— Accurate theory, – – – Approx. theory

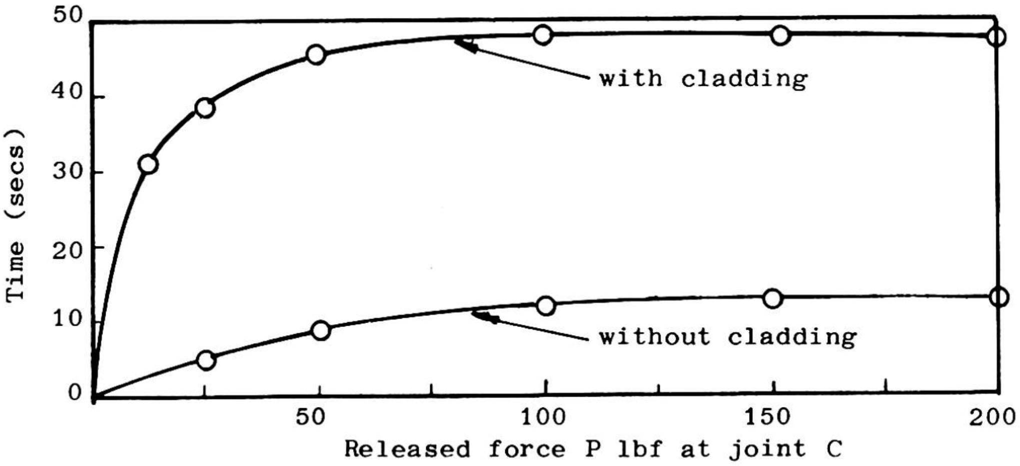
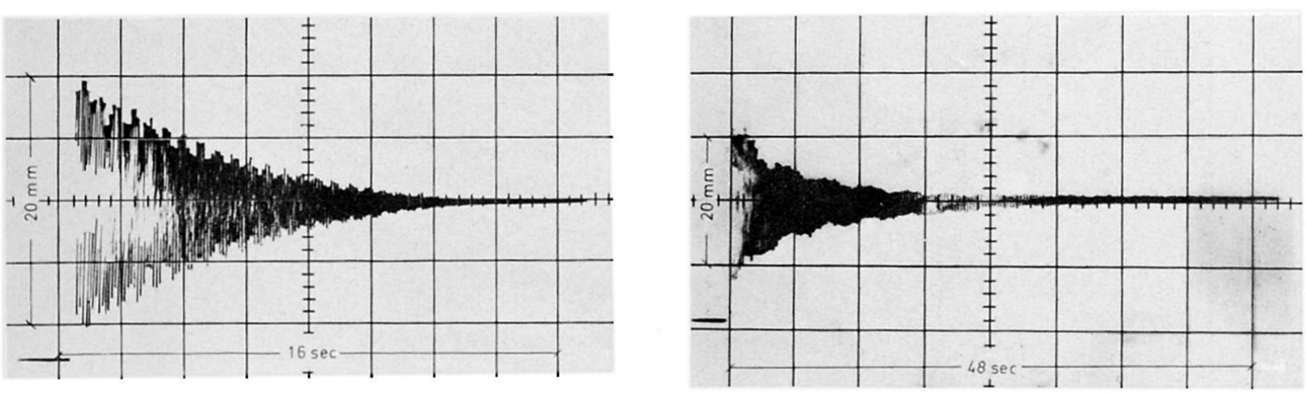


Fig. 4

Time for vibrations to cease for increasing initial forced displacements



(a) Vibration without cladding, P=150 lbf (b) Vibration with cladding, P=150 lbf

Fig. 5.