Enclosed Network Bridging Structures for Urban Environments

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ABSTRACT

The paper presents the results of a computational assessment of prestressed cable networks braced against slender hoop bearing systems to form wide span bridging structures; these may be classified as hoop tensegrity structures. The main subject of the study is a continuous multi-span network bridge system suitable for enclosed transport from suburban to more central city areas; the example chosen is a street scale network bridge for pedestrians, cyclists and small electric vehicles. A less efficient single bridge with only one main span and two cantilever spans is also considered. The performance of the system is assessed in relation to typical code requirements and compared briefly with an equivalent conventional boxed truss structure.

Key Words: Tensegrity, Network, Bridges, Funicular Polygons

1.   INTRODUCTION

The structural systems considered in this paper are closely related to hoop tensegrity physical models studied by Robert LeRicolais, which he termed "Funicular Polygons of Revolution" (figures 1a and 1b). [1]
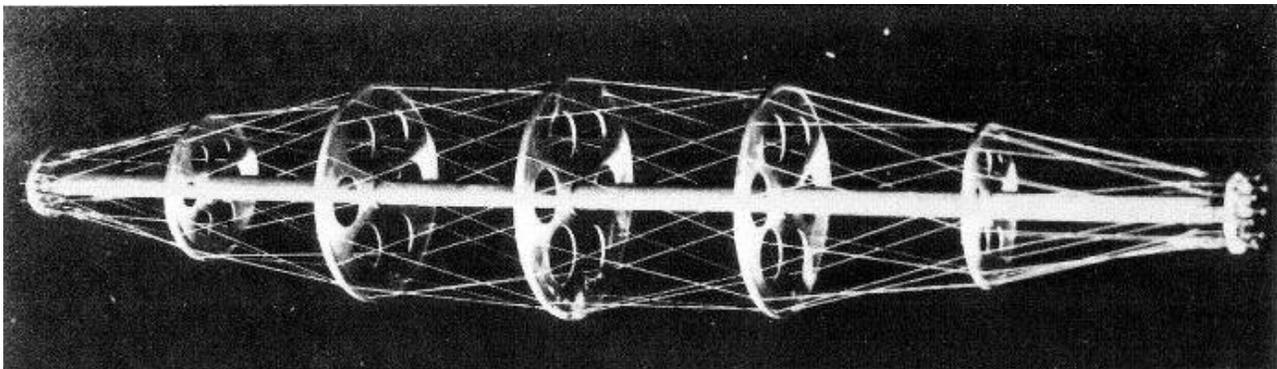

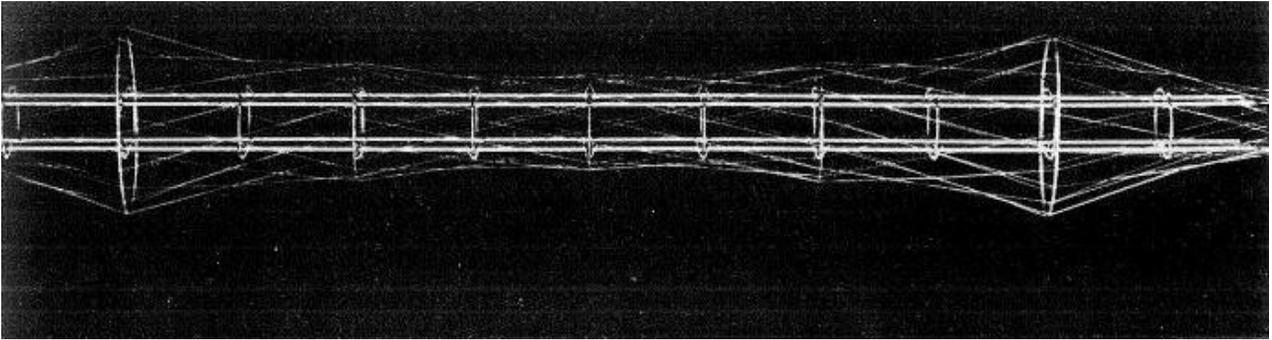Figure 1a (and b) : Physical models of Funicular Polygons of Revolution.

Figure 1b

Le Ricolais also envisaged that such structures might be used for transport in cities [1]: "…the ground surface is totally occupied …so the system we have proposed would consist of a starhex network of aerial bridges spanning about 1600 ft between interchange stations…..the tensile forces of each span would be balanced by the tension in adjacent spans, the ultimate force being absorbed at the ends of the line in suburbia. …….With a system of elevated transit we could perhaps open up a new dimension – the aerial view of the city as a whole" (as shown in figure 2) [2]. That is a rather startling scale, and perhaps as Le Ricolais noted somewhat "utopian"; he also said "I wish a model could be made because I'm sure it would help us to detect what is wrong and what might take place".
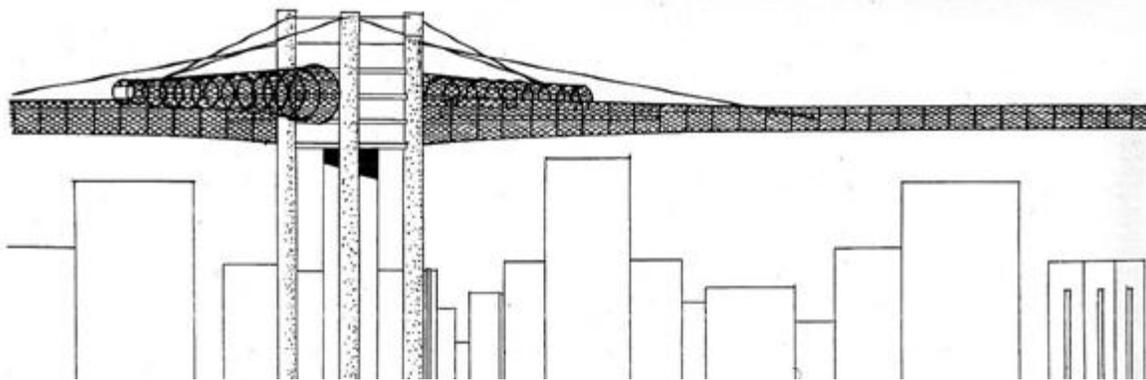

Figure 2: Schematic drawing by Le Ricolais of a funicular polygon of revolution in an urban centre.

That is the context of the present paper, but the scale chosen for the computational study models is significantly smaller in both span and height – typically represented by the initial single core model (rendered in figure 3) which has a total span between its "free" ends of about 500 ft.   This model, although functionally merely a pipe bridge, was useful in evaluating the extent to which bending moments caused by design loads could be sustained solely by variations in the network cable tensions, with compression in the central core remaining practically constant. It was also used to assess appropriate shapes and properties for the hoops, and the arrangement and sizes for both the external bounding cable network and the internal bracing cables connecting the hoops to the main boom.

The reason for starting with the assessment of a simple free end network bridge rather than a continuous system (the main objective of the paper) is that any large heavily populated multi-span system should be resilient to sequential collapse – for example, capable of sustaining catastrophic failure (by sabotage or extreme natural causes) in one or more of its spans with all other spans of the

structure remaining stable under full loading. Thus in such an event one or more spans becomes truncated with free end(s), evidently subject in adjacent areas to abnormal local loads.
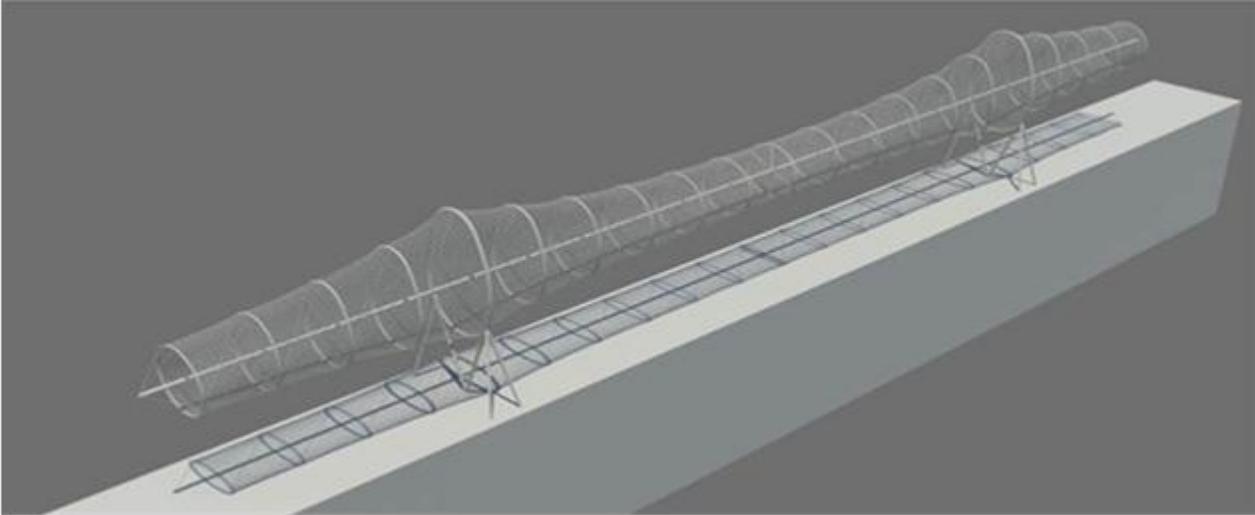

Figure 3: Initial single centre boom model with centre span of about 300 ft and side spans of 100 ft.

The principal study models in this paper have twin booms which allow for proper internal use and are dimensioned to simulate street scale enclosed bridges for pedestrians and cyclists (and also electric buggy vehicles of about 4 ft (1.2 m) width). The dimensions of the structures allow for 305 ft (93 m) spans with two 10 ft (3 m) tracks on one level. At the main support nodes (and potential entrance areas) a second level can allow for small meeting, market or rest areas. Two systems were assessed: a free-end bridge and the main continuous multi-span bridge system. The analysis is performed by Dynamic Relaxation ([3],[4],[5]) which can be accurately applied for both form studies and engineering analysis, including the modelling of resilience to collapse.

2. STRUCTURAL DESIGN CRITERIA

2.1. Stress/Strength Requirements and Design Loads

In most international codes of practice structural engineering analysis is based on the concept of limit state design. For example serviceability limits for maximum deflections under working loads and ultimate limit states to guard against structural collapse. For different types of loading different load factors are applied (eg 1.2 for dead loads, 1,4 for wind and 1.6 for imposed loads), and different reduced factors also for the various load combinations. In simple terms, for strength assessment, structures subject to these factored loadings should sustain stresses which are less than yield or failure stresses in all parts (allowing for reductions due to material imperfections and stability effects). Applying this approach in a paper intended as a feasibility study is unnecessarily involved, and for this reason a simpler stress and strength factor approach is used in which the unfactored full design loads (and combinations) are applied and stress/strength factors are used to assess the security of the system.   For steelwork such as the bearing hoops and support struts a stress factor of 1.5 is used, for the compression booms 1.8, and for cables a factor of 2.5 against minimum breaking strength is used. The steel is assumed to be grade 55 with a yield stress of 450 N/mm^2 and the cables 1x37 spiral strand with nominal breaking strength 1570 N/mm^2.

The imposed design loads for pedestrian bridges (including loads for cyclists and small electric vehicles) are, internationally, typically between 4 & 5 kN/m^2 with snow loading being effectively included in this figure since most pedestrian bridges are uncovered. In the present study imposed loading of 4 kN/m^2 over the two lane width of 6m is used together with an allowance of 0.8 kN/m^2 over the projected plan area for snow loading; accounting for slope on the surface of the structure that equates to a basis / ground plan snow load in excess of 1.6 kN/m^2.   If the internal environment is assumed to be partially heated and the covering surface smooth (for example ETFE foil cushions supported from the bounding cable net) the latter figure for snow loading is reasonable; but clearly that (and other environmental loads such as wind loading) would be dependent on location.   To account for two level use at the main support areas an additional 30 tonnes of imposed loading is distributed in these areas.   The self-weight or dead load is based on the component weights of the structural steel and cables and includes also an allowance of 3 kN/m length of the bridge for non-structural components such as deck coverings and lightweight cladding (the latter would be covered by approximately 0.3 kN/m length).   The wind loads used are based on a design wind speed of 100 mph (44 m/sec) at mean structure height with a structure drag coefficient of 1.35 x projected area. Obviously these are simplified design loading conditions without regard to specific location.

In addition to the above, for the multi-span system in particular, assessments are made for catastrophic loading conditions with full imposed loading on one span and adjacent spans failed, and in the normal state for appropriate ranges of loading under temperature variations of +/- 32° C.

2.2. Deflection Limits

The various international recommendations or required limits on deflections for pedestrian bridges range from 1/250 span to 1/1000 span, with the latter figure more appropriate for highway bridges with pedestrian sidewalks. Common figures used for pedestrian only bridges are 1/360 span (Canada and Australia), and 1/400 – 1/500 span (USA and UK), with a lower figure of 1/220 span for cantilever bridge spans. These figures are intended to apply under all conditions of loading; the most severe generally being vertical deflections under full uniformly distributed imposed loading together with lateral deflections under full wind loading. For multi-span systems the most severe state is generally uniform loading on one span with adjacent spans unloaded. For the current study of enclosed bridges the most severe state is full imposed load on one span together with full snow loading throughout. The target deflection limit under these multiple conditions is 1/400 span. The dynamic response of the study bridge models has not been assessed, except in the crudest context of compliance with these conventional deflection constraints.

3. THE NETWORK HOOP TENSEGRITY BRIDGE MODELS

The studies were undertaken in the following order: Core bridge model (figure 3), Twin boom and free end bridge model (figures 4 - 7) and the main Symmetric multi-span model (figures 8 – 12). There was a gradual refinement of the structural properties such as hoop, boom and cable sizes throughout this process, ending with the lightest weight properties for the multi-span system to comply with the target deflection and stress constraints.   These properties were then re-applied in the less efficient free end bridge systems in order to make brief relative comparisons; the initial systems do not therefore comply with the target constraints, although means to improve their performance are discussed.

3.1. Twin boom free end bridge

Figure 4 shows an elevation of the numerical model used, in which half the structure is modelled since symmetry can be imposed about the centre plane (at left hand side of model). The dimensions of the model are: length 82 m from centre to free end (RHS), maximum height 25 m, minimum (central) hoop diameter 7.5 m, maximum hoop height and width 16 x 9.5 m (for the two main support hoops – see figure 5b), and separation between boom centres 3.75 m.
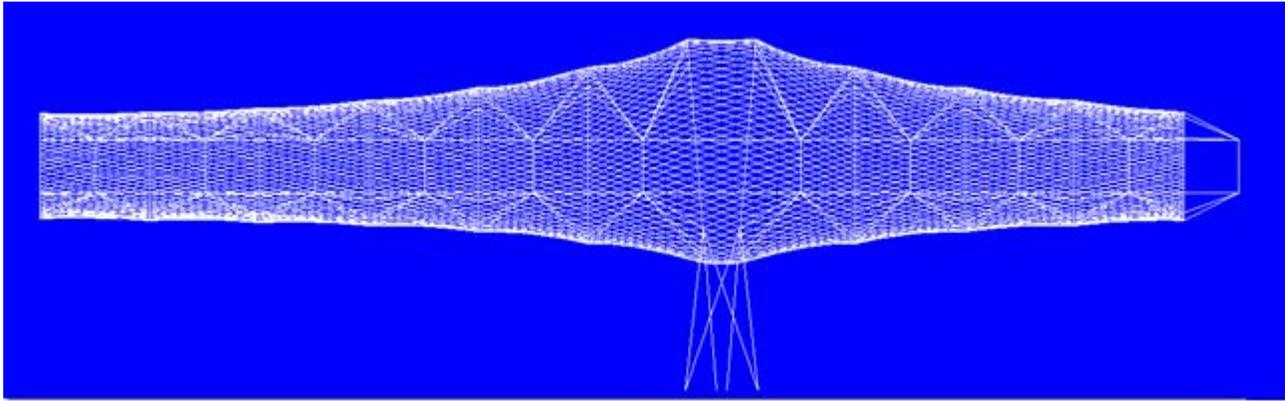

Figure 4: Elevation of a Twin boom free end bridge, system is symmetric about LHS extremity.
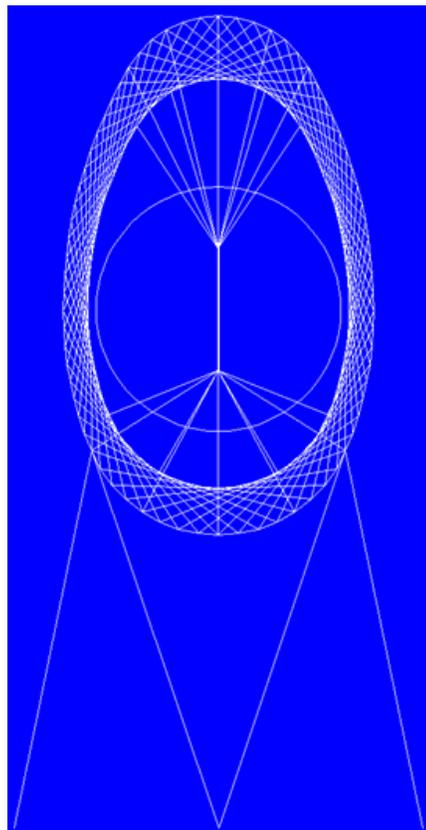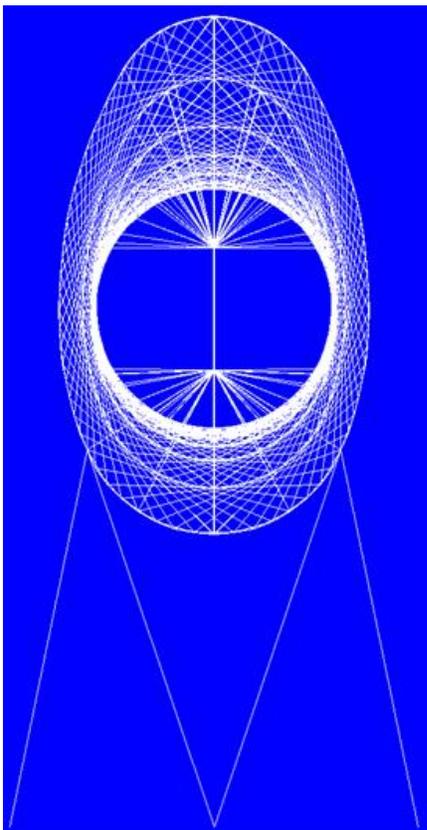

Figure 5a)   View along whole bridge   Figure 5 b)   Section at main support hoop.

Excluding the two splayed main hoops, in the left part of the model there are 6 CHS hoops: 4 of 203 x 11 mm, 1 of 244 x 12.5 mm and, nearest the support hoop, 1 of 305 x 16 mm. In the right hand part there are four hoops of corresponding section sizes. The two splayed main support hoops are CHS of 419 x 25 which are additionally stiffened at the junction with the support tubes (all of 305 Φ section); that size of section, or an equivalent truss, being necessary to accommodate the very high wind forces which must be equilibrated to ground at this location. The internal bracing cables are all 28 mm diameter, and the bounding network cables two sets of 72 x twin 16 mm diameter. These would require saddle clamping supports of 250 mm radius and 100mm length (to accommodate a maximum angular deviation of 25° over the hoops). Both of the main compression booms are CHS of 457 x 20 mm.

Prestressing of the complete network was achieved by imparting a constant force in each main boom of 6 MN. Physically or practically this would imply jacking at both ends of each boom (by approximately 18 cm. A the same time the diagonal internal bracing links must not be fixed to the main booms but attached on sliding collars, and subsequently fixed only after the prestressing (the variations in forces along the lengths of the booms in all service loading cases are small). At the free end of the system the end hoop and diagonal bracing cables connecting to the ends of the booms are greatly stiffened to accommodate the boom forces. Also in the section between the two end hoops a distributed counter dead-weight of 18 tonnes is applied. All of these prestressing actions are simulated in the numerical modelling process, and together they impart a pre-camber in the centre span which counters the dead weight of the complete system.

An enlarged elevation at the main support location is shown in figure 6, and plan view in figure 7; these figures also apply to the continuous network bridge covered in the following section (3.2). The arrangement of internal bracings permits two level occupation in this area. Note that the main bridge deck (which is not shown in any of the figures) could be balanced propped cantilevers from the lower boom or alternatively longitudinal stringers supported on beams spanning transversely at each hoop. The weight of either of these, together with light decking, is covered by the self-weight allowance.
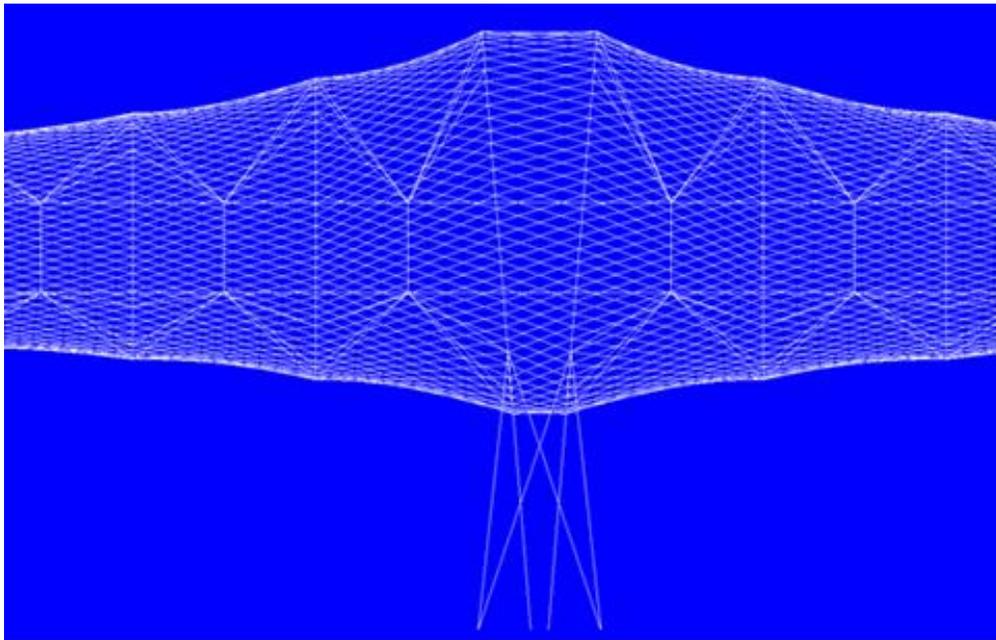

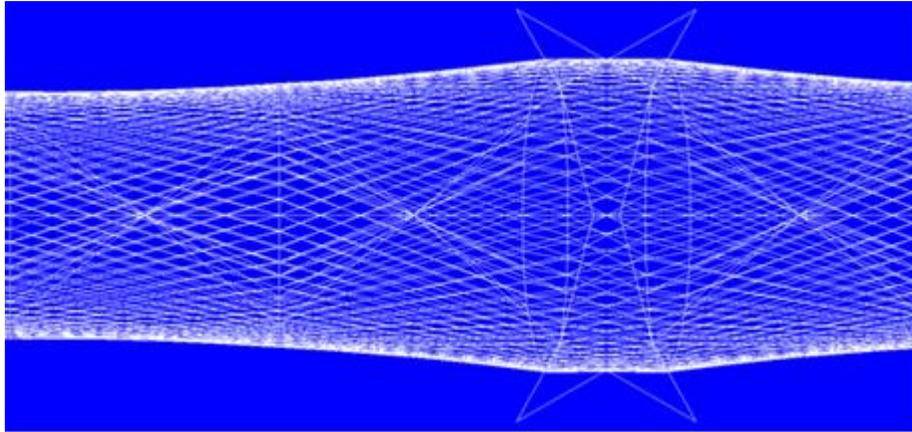Figure 6: Enlarged elevation of main support.

Figure 7: Plan view of main support.

3.2. Symmetric model for multi-span bridge

The continuous multi-span bridge model is shown in figure 8, and the structural properties previously given were applied in (and developed for) this bridge system; the only exception being the main boom properties with the upper boom increased to 457 x 25 CHS and the lower boom decreased to 343 x 16 CHS. Figure 8a shows a rendered view of the symmetric model reflected about one centre plane (although of course it is intended as a multi-span system).
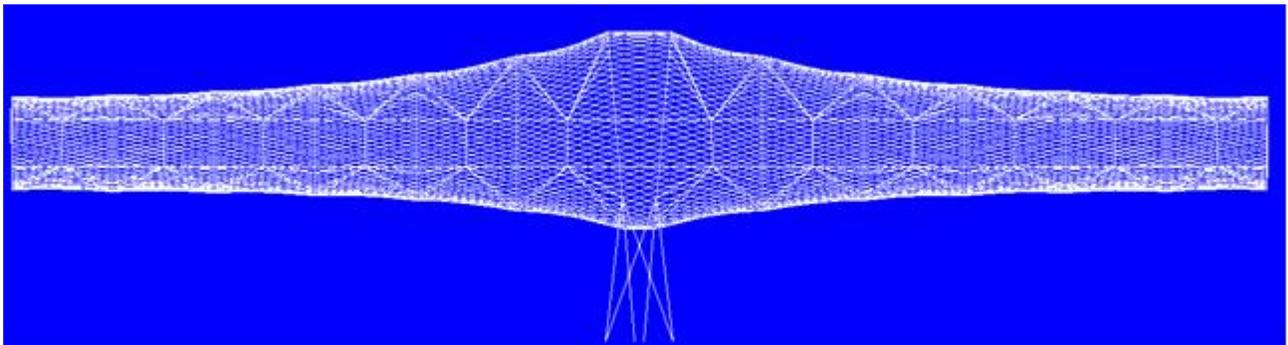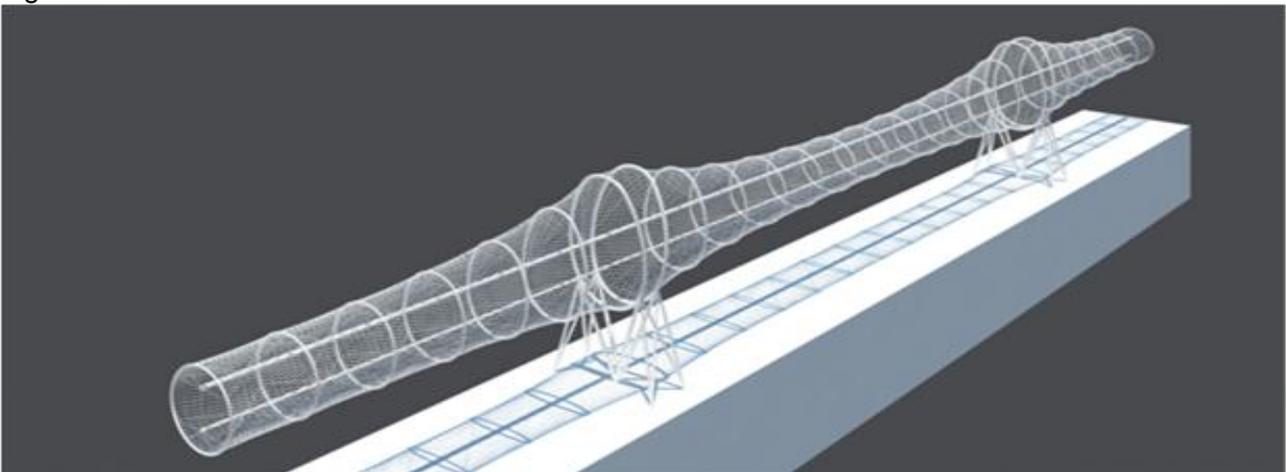

Figure 8:   Elevation of one section of a multi-span model

Figure 8a

Constant prestressing forces of 8 MN in the upper boom and 4 MN in the lower boom were imposed. Physically the stressing out process would be far more complex than for the free end system since, in addition to pre-compressing the main booms (requiring a jacking of approximately 10 cm in each 93 m span), the bounding network would need to be independently prestressed. This might, for example, be achieved by tensioning together the two splayed main hoops, and this would also impart a pre-camber in each span. But, more likely, this could alternatively be achieved by gradual jacking of each network cable pair at the hoops. Note that the bounding network cables are terminated in each span at the main support hoops, but otherwise they are continuous throughout each span – of course clamped on saddles at the hoop locations. The symmetric model used for analysis can simulate most important loading conditions – either uniform or non-uniform with one span fully loaded and adjacent spans unloaded or partially loaded.

4. ANALYSIS AND RESULTS

For clarity, the main presentation here relates to the continuous bridge system; comparative results for the free end bridge structure(s) are given at the end of the section.

Figure 9 shows a view along the bridge under the condition of prestress + self-weight; at the centre span positions there is a slight sag of about 4 cm, but this could be corrected by adjusting the boom forces to provide some pre-camber. Figure 10 shows the same view for the most severe state of deflection caused by imposed loading on one span (with adjacent spans unloaded) together with full snow loading throughout. The maximum deflection from the prestress + self-weight state is 0.228 m or 1/409 span, and it was this state with a target of 1/400 that was used to establish the required cable sizes and prestressing. For an unbalanced load case of 4 kN/m^2 on one span with adjacent spans unloaded (ie no snow loading) the maximum central deflection is 1/460 span.
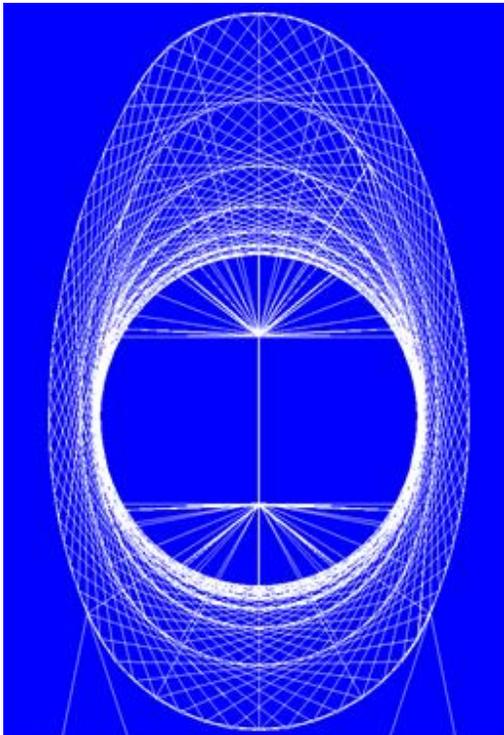

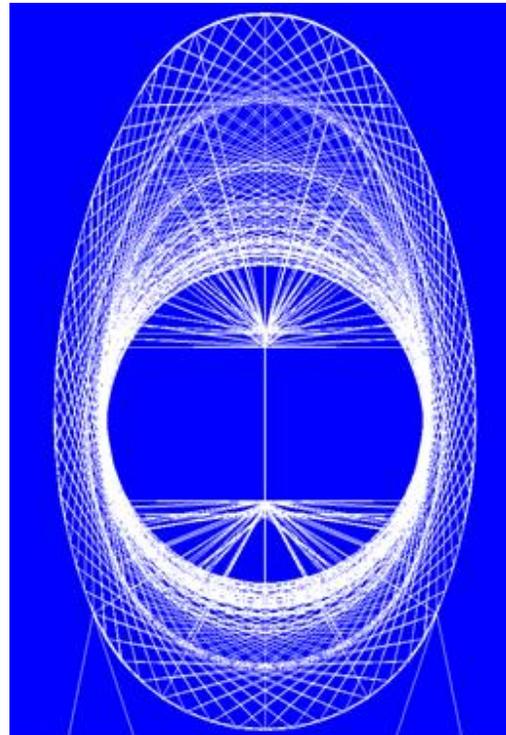Figure 9: Prestress + Self weight   Figure 10: Full imposed on one span
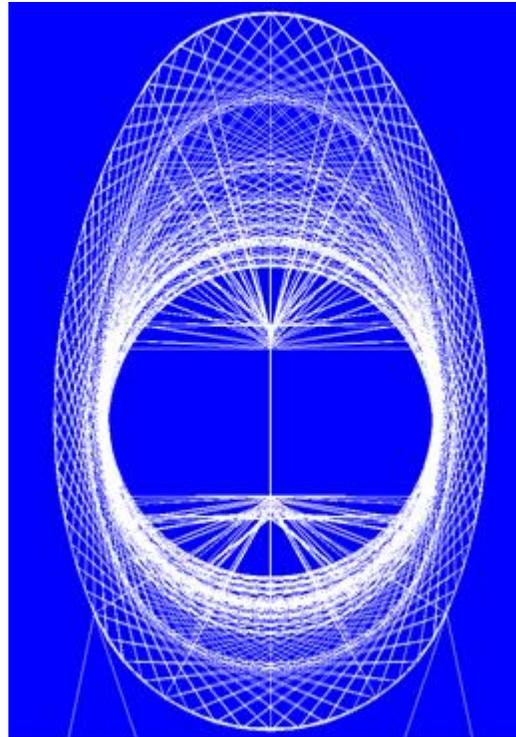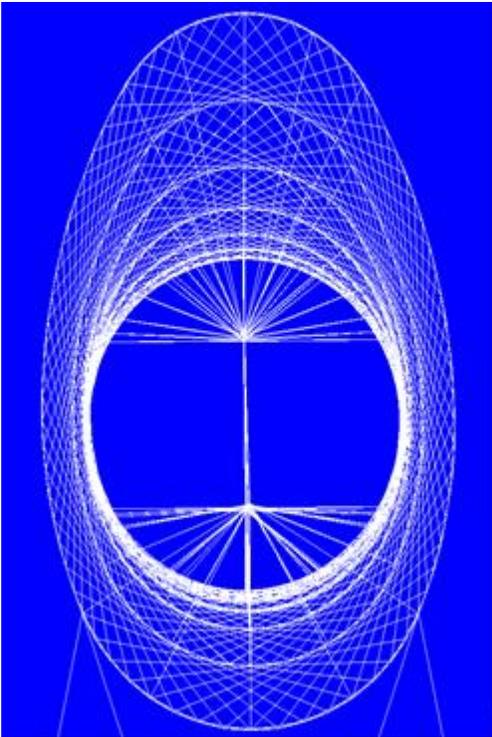
Fig 11: Full uniform imposed + wind load   Fig 12: Imposed with failed adjacent spans

One of the most severe conditions for strength assessment is the combination of uniform imposed loading + maximum wind loading throughout. Under this condition the maximum stress in the top boom is 236 N/mm^2 and the maximum tension in the bounding cable network is 148 kN – this is shared between two 16 mm Φ cables, each with a minimum breaking strength of 206 kN.   The deflected state view is shown in figure 11; the maximum vertical displacement is 1/896 span and the maximum lateral displacement (due to wind loading) is 1/659 span.   This state with 4 kN/m^2 imposed load + 44 m/sec wind load assumes that any snow would be blown off the structure. A more severe state for vertical deflections is the full imposed + snow loading; for this condition the maximum deflection is 1/742 span (and maximum boom and net forces are very similar to those quoted above).

Temperature effects were assessed on the assumption that there would be some control of the internal environment, and particularly that the range of temperature variations of the main booms could be controlled. The following figures were therefore used for assessments: increase / decrease in temperature of: bounding network, hoops and support members +/- 32° C; internal bracing cables +/- 24° C; compression booms +/- 8° C.   The most severe serviceability condition for maximum deflections is full temperature increase with one span having 4.5 kN/m^2 imposed loading and adjacent spans unloaded (the increased imposed loading is to allow for additional fittings loads – obviously there is no snow loading). Under this condition the maximum deflection is 1/410 span. The condition is also the most severe state for maximum compression in the top boom – the stress increases to 266 N/mm^2; this represents a Factor of Safety on yield stress of 1.7 rather than the target 1.8. Decrease in temperature (-32° C) has the effect of increasing the network tensions (but obviously reducing deflections and the boom compressions). Under this state the maximum net tensions increase to 166 kN, which is only slightly below the target of 165 for a Factor of Safety of 2.5.

In the event of sudden failure in any span very high loads must be transferred from the main booms via the hoop bracings to the bounding cable network. This is the principal reason for using diagonal bracings; studies of the initial core model showed little difference in stiffness between a

system with diagonal bracings and one with vertical bracings in the planes of the hoops, but clearly the latter would be inadequate in the event of local failure in the continuous system. The choice of failure mode is obviously indefinite, but for simplicity a single mode has been assessed, with one span having full internal imposed loading and adjacent spans failed at mid-span and being unloaded except for self-weight. In that state the boom forces and network forces at mid-span reduce to zero, but through transfer over several hoop bays and the support system must develop adequate forces to safely sustain the loaded span(s). In this extreme situation it cannot be expected that serviceability stress and deflection targets will be met, thus safety requirements might be based on ultimate manufacture quoted strengths rather than factored strengths. The analysis showed a maximum downward deflection in the unfailed span of 0.28 m or 1/330 span (see Figure 12). This compares with a deflection of 1/460 for the unfailed structure subject to the same imposed loadings and demonstrates adequate serviceability of the system (at least for the one quite severe failed state considered). In the failed span there is an upward deflection of 0.23 m at the terminating mid span position, where the hoop forces and moments are zero. However, the next hoop suffers a predicted maximum stress of 625 N/mm^2, and would clearly yield. If the local failure moment in this hoop (which is a combination of both in-plane and out-of-plane moments) is re-distributed to the two adjacent network node positions (ie over a distance of 0.65 m) this localised stress reduces to 421 N/mm^2. The bracing links connecting this hoop to the main booms undergo a severe increase in tension up to a maximum of 688 kN (and minimum of 615 kN). For 28 mm Φ cables that would require steel of grade 1770 N/mm^2 (minimum breaking load 711 kN) or preferably 30 mm Φ cables (816 kN). A similar check on the next hoop (away from the failure position) shows a maximum hoop stress (without redistribution) of 385 N/mm^2, and bracing cable forces well within the ultimate capacity of 28 mm Φ cables. Subject to the small revision of bracing cables the security of the structural system in this one failed state thus appears acceptable (or needing only small design changes).

4.1. Comparative performance of the Free end Bridge

Using the structural properties and prestress regime previously quoted, under self-weight alone the initial pre-camber in the Twin Boom Free end Bridge (fig 4) is + 2 cm. Under imposed and snow loading uniformly distributed over the whole structure the centre span deflection is 0.212 m or 1/440 span; this compares with 1/742 for the continuous system subject to the same load state. Under the more critical loading condition for deformations, with the centre span fully loaded with imposed + snow and the side spans with snow alone, the central deflection is 0.291 or 1/320 span. With the net cables increased to 2 x 18 mm Φ this reduces to 1/355 span.

The initial "core" bridge with a single boom (Figure 3) is much more flexible (and impractical). Using the increased size for net cables of 2 x 18mm Φ and a boom of CHS 457 x 40mm the mid span deflection under full imposed & snow loading is 0.306 m when using vertical internal bracings (in the plane of hoops) and 0.295 m or 1/305 span with inclined bracings; in the former case the whole of the shear loadings are transferred effectively to the bounding cable net. In the worst case for deformations, with only the centre span fully loaded, the mid-span deflection is 1/175 span.

4.2. Comparison with a conventional box truss bridge

For the continuous structure the total weight of the bridge (figure 8) over each span of 93 m is 1460 kN, and of this total, the external support tubes weigh 137 kN. Neglecting the weight of these support tubes, the equivalent distributed weight is 14.2 kN/m length of the bridge and this figure is used for a comparative box truss bridge. The box truss bridge is assumed to have main tubes at its corners which are 419 x 20 mm CHS with the overall cross section of the box truss measuring 4.5 m deep x 6.5 m wide between these tube centres. Together with appropriate shear and cross bracings and a lightweight 6m width of decking + lightweight cladding (eg ET foil cushion) the self-weight would

be about 14 - 14.5 kN/m. Using this as a standard for comparison, the strengths of the two systems under the various conditions of loading are similar but deflections of the conventional bridge are significantly less – for example for the unbalanced imposed load case (alternate spans loaded) the deflection would be about 1/600 span compared with 1/460 for the network bridge (these figures assume similar support arrangements).   One aspect which for purposes of comparison mitigates for the network bridge system is that the two main support hoops (including stiffeners at the support tube nodes) weigh 237 kN. The principal reason for the large size and weight of these particular tubes is the height of the structure and the fact that it is enclosed and must sustain high wind loads. In both systems, the network bridge and the box truss, the same low allowance has been made for self-weight of decking and cladding (approximately 3 kN/m length). This may not be adequate to cover additional fittings loads, which is the reason why an extra allowance of 0.5 kN/m2 was made when considering the most severe serviceability condition for the network bridge (of full imposed load on one span only together with maximum temperature increase).   Note that temperature stresses in the box truss bridge would have to be alleviated, for example, by using a construction with expansion joints at the quarter points of alternate spans.

5. CONCLUSION AND FURTHER STUDIES

On the basis of these restricted studies the continuous network bridge would seem to be a viable and reasonably efficient structural system. Because of the use of high tensile steel cables with greater flexibility than normal steel the most severe design constraint is the serviceability limit state on deflections, although there may be consequential gains for ultimate strength states. Also in this respect the hoop supported cable net system should have advantages in resilience to shock or earthquake loadings – which is a normally accepted advantage of suspension bridges.   The adequacy of the catastrophe / failure state analysis (with the system severed at mid-span) is also encouraging since this was originally expected to be a potential downfall of the entire system. The main criticism of the network bridging structure is the potential complexity of the structural details and stressing out procedures. These issues will need to be assessed in collaboration with consulting structural engineers.

In further concept studies it is intended to consider the design and feasibility of branched network bridging structures of larger scale – for example the system sketched in figure 15 with spans about half of those proposed by Le Ricolais.
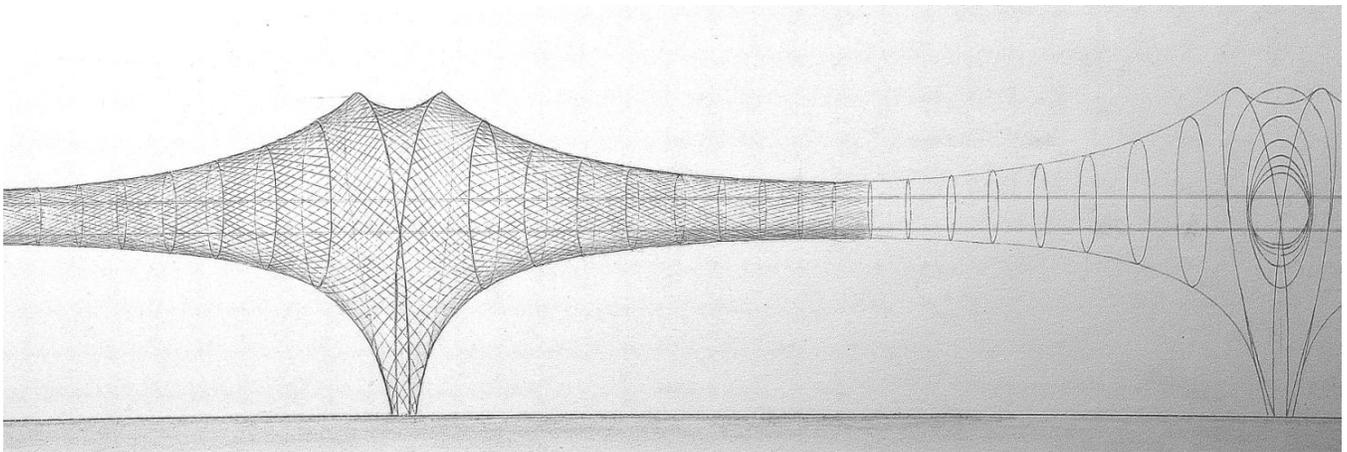

Figure 15: Proposed large scale study model 800 ft (245 m) spans with branched support nodes

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