Stability of a Bowstring Bridge with Twin Inclined Arches

Stabilité d'un pont bowstring avec arcs inclinés, contreventés en clé

Stabilität einer Bogenbrücke mit zwei geneigten und am Scheitel verbundenen Bogen

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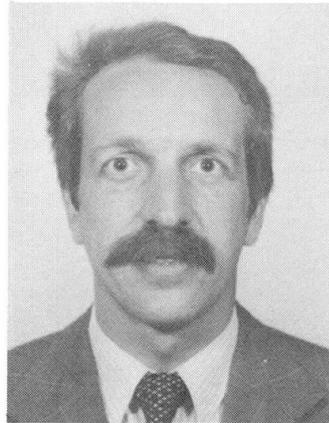
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SUMMARY

Several finite element analyses, of increasing sophistication, are used to analyse the stability of a special bowstring bridge: the deck is carried by a widely spaced lattice of hangers, the arches are inclined and connected at their crown by a single bracing strut. The computations emphasize the importance of correctly simulating the role of this strut. if done so, plane finite elements can be used to model the three-dimensional behaviour.

RESUME

La stabilité d'un pont métallique bowstring d'un type particulier (les deux arcs, à suspensions croisées, sont inclinés et liés en clé par une seule entretoise) a été étudiée au moyen de divers types d'éléments finis de sophistication croissante. Les calculs montrent l'importance d'une simulation correcte de la liaison en clé. Sous cette condition, des éléments finis plans permettent d'obtenir une bonne approximation du comportement tridimentionnel.

ZUSAMMENFASSUNG

Die Stabilität einer metallischen Bowstring-Brücke von einem besonderen Typ (die beiden Bogen mit gekreuzten Hängestangen sind geneigt und am Scheitel mit einem einzigen Querträger verbunden) wird mit immer komplizierteren finiten Element-Modellen analysiert. Die Computerberechnungen zeigen die Wichtigkeit einer korrekten Simulierung der Scheitelverbindung. Nur so erlauben es ebene finite Elemente das dreidimensionale Verhalten gut zu erfassen.

1. INTRODUCTION

The King Albert Canal which links Liège and the port of Antwerp is being wide-
ned to allow traffic of 9000 T pushed barges. This requires replacement of se-
veral bridges, among them the one at Hermalle with a main span of 138.10 m.
Choice was made of a steel bowstring bridge carrying the deck by means of a
widely spaced lattice of hangers ; the two box girder arches are unusual because
they are inclined with an angle of 9.44° degrees and connected at their crown
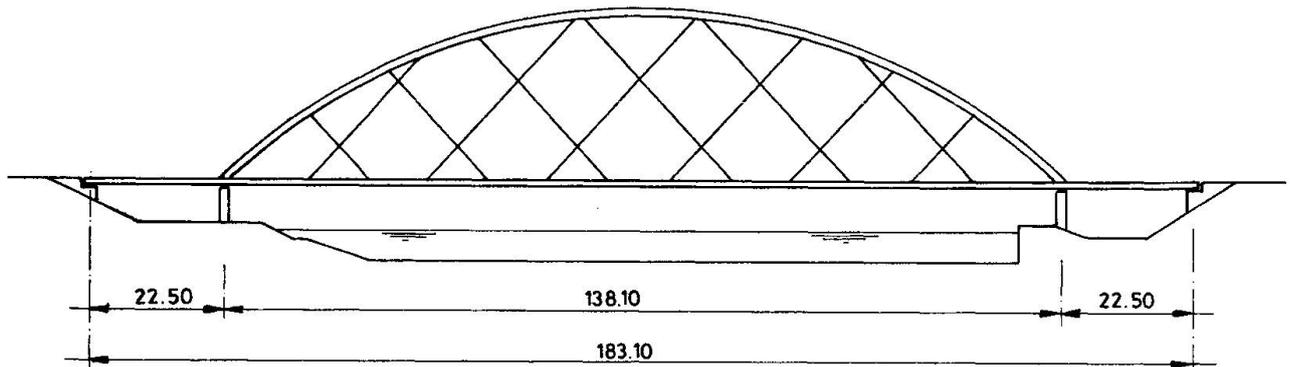by a single bracing strut. (Fig. 1).

Figure 1 Side view and cross section of the bridge

Simple calculations have shown that in-plane insta-
bility is not to be feared. But out-of-plane instabi-
lity of the arches must carefully be studied taking
into account the appropriate components of the han-
gers forces acting on the arches and the elastic
support provided by the short bracing strut at
their crown.

A nonlinear finite element program, especially well
suited to metallic structures, has been developed
at the University of Liège by the Structural Divi-
sion of the Civil Engineering department [1], [2],
[3], [4]. It has already been successfully used
for several research projects [5],[6].

The purpose of the present contribution is to show
how the University and the consulting bureau, wor-
king hand in hand, have used this program and increasingly sophisticated finite
elements to predict the behavior of the arches with increasing accuracy, while
minimizing computation costs which could have skyrocketed if the most advanced
techniques had indiscriminately been used.
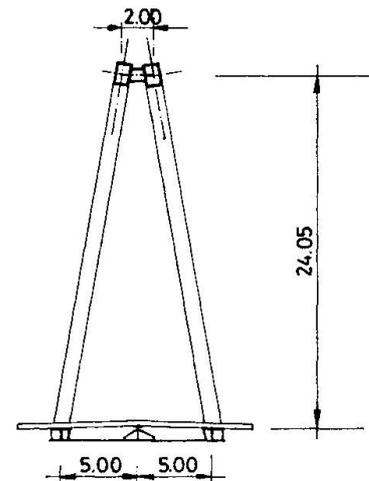
2. STEP-BY-STEP COMPUTATION OF AN ARCH IDEALIZED AS A STRAIGHT BEAM.

2.1. Idealization.

For the preliminary design, one wished to use only the simplest version of the
non linear program and   plane beam finite elements. To do so, one single arch
was developed and the hangers were idealized as elastic supports.

The stiffness of these elastic supports is due to the fact that, under a lateral
displacement of the arch, the hangers become inclined with respect to the origi-
nal arch plane; this induces a non linear pull-back force which is simulated by
means of springs the stiffness of which is modified according to the arch dis-
placements (Fig. 2).
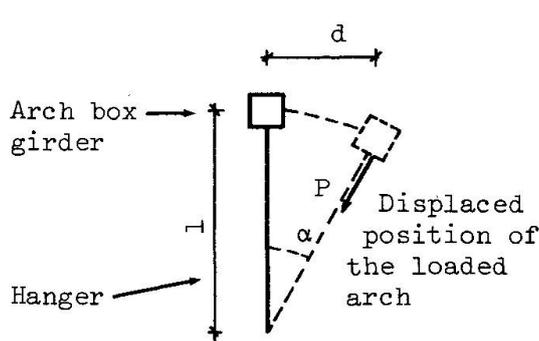
P = force in the hangers
d = out-of-plane displacement of the arch
  = l.tan(α)
F = pull-back force on the arch due to P
  = P.sin(α)
k = stiffness of the spring idealizing the
    pull-back force
  = F/d
  = P.sin(α) / l.tan(α)
  ≃ P/l : stiffness varying as force P
          with α << 1 rad

Arch box —▶
girder

Hanger —▶

Displaced
position of
the loaded
arch

Figure 2 Pull-back forces of the hangers

The idealized structure is shown on fig. 3.a ; the springs are vertical bars on
the drawing ; the two stubby beam elements at the ends of the developed arch si-
mulate the restraints caused by the floor beams at the arch ends. In this first
approach, *the central node is considered as fixed against transversal displace-
ments to simulate the crown bracing.*

2.2. Loading.

The loading is considered to be proportional even though it actually results
from the superposition of dead and live loads. The distribution of the axial
forces in the arch is obtained by means of a plane   linear elastic analysis.
These axial forces are then induced in the fictitious straight beam by means of
appropriate external loads applied to its nodal points ; their longitudinal dis-
tribution is supposed to retain its overall shape while their intensity varies,
because in-plane deformations of the arch are very small.

These simplifying artifices thus enable to use plane beam finite elements to
study the out-of-plane instability of an arch loaded in its own plane.

Two successive deformed shapes corresponding to increasing load steps are shown
in fig. 3.b and c : of course an antisymmetrical mode does appear.

Load-displacement curves for a typical point of the arch are drawn in fig. 4.
They exhibit the instability by divergence behavior undergone by the out-of-
plane displacements. It must be noticed that, under wind load (curve c), out-
of-plane displacements appear very soon because of the presence of lateral for-
ces but these same forces oppose one of the half sine   instability wave and res-
train the intensity of the displacements.
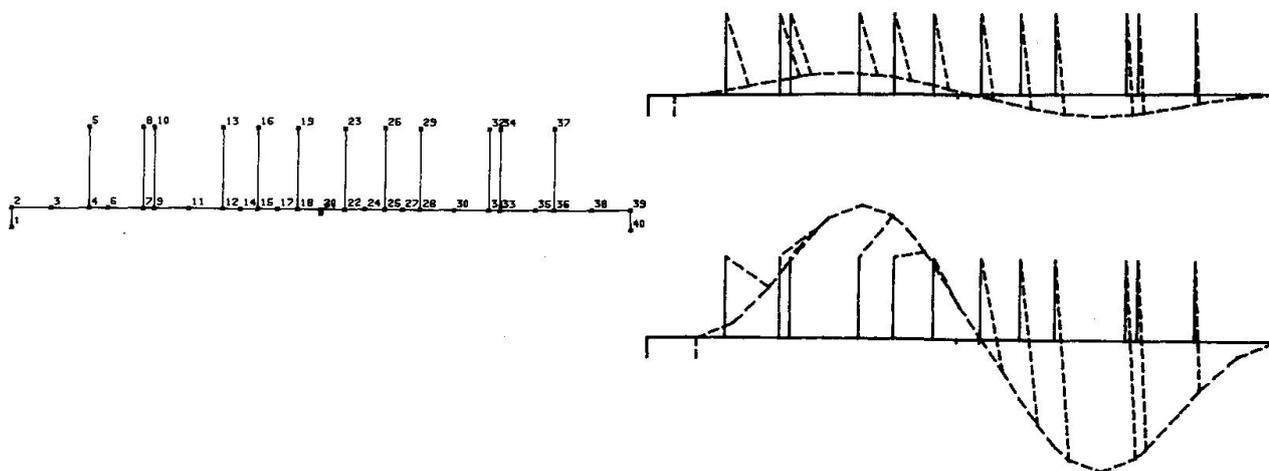
Fig. 3 Idealization of an arch by means of a fictitious straight beam and
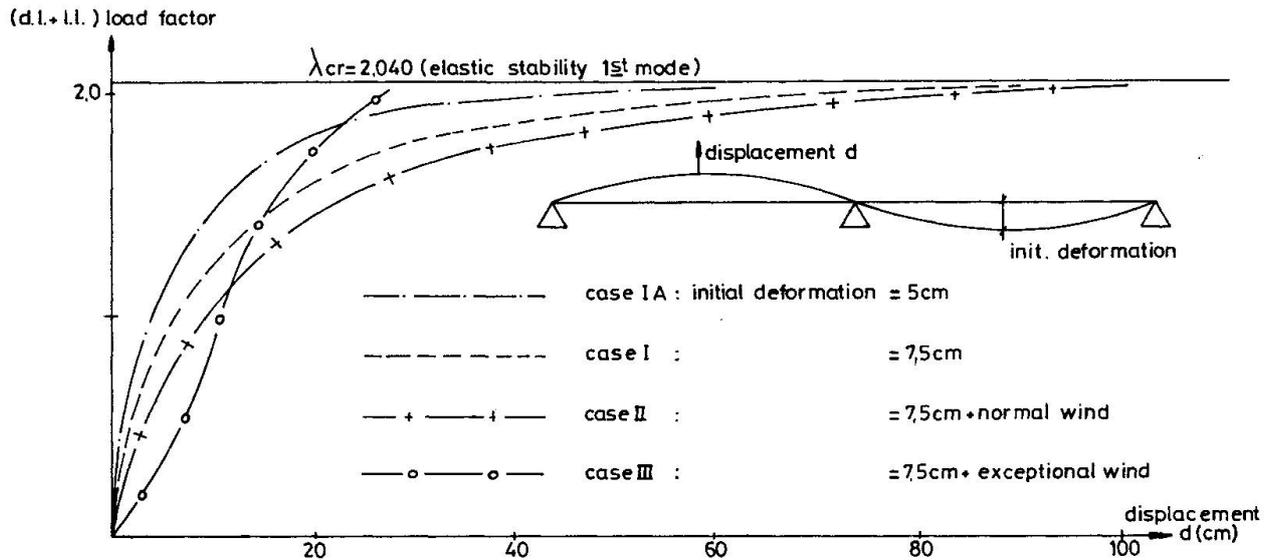   typical deformed shapes.

Fig. 4   Load factor versus out-of-plane displacement curves under different
         step-by-step loadings.

3. STABILITY COMPUTATIONS FOR A SINGLE ARCH.

Considering the important approximations made in this simplified analysis, a
checking was deemed necessary even for the preliminary design. As a verifica-
tion, the critical load was computed for a single arch *simply supported at is
crown*. This time the idealization used space beam finite elements and truss
bars (the prestressed hangers always remain under tension and are considered
as truss bars with an adjusted elasticity modulus); the idealization is shown
in fig. 5.

A linear elastic computation enables to form the structural stiffness matrix K_0
and to find the necessary stresses to form the initial stress matrix K_σ; from
this, assuming again a proportional loading, one can find the critical load fac-
tor, by means of

$$\det [K_0 + \lambda K_\sigma] = 0$$

The first two critical load factors and the corresponding instability modes are
shown in fig. 7. The lowest critical value of λ is also represented as a hori-
zontal line in fig. 4 ; it perfectly confirms the results of the incremental non
linear analysis. This excellent agreement can be attributed to the smallness
(and hence the weak influence) of the displacements under loading prior to out-
of-plane instability.

On the other hand it is rather disappointing to discover that the first critical
load factor for the arch with a simple support at the crown is little different
from the first critical load factor for the arch without a crown support as
shown in fig. 6. This is due to the pull-back effect of the hangers : they
strongly oppose a single wave instability mode because they slope at a larger
angle than in the case of a double half wave. In such bowstring bridges, the
crown bracing strut seems to be of little benefit. But, until now, *one has
neglected the rotational restraint due to this crown bracing.*
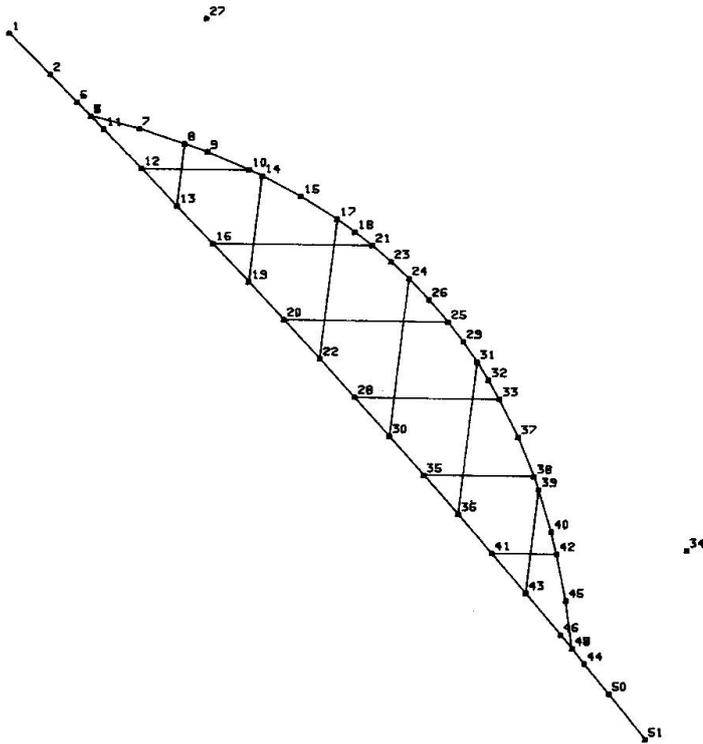
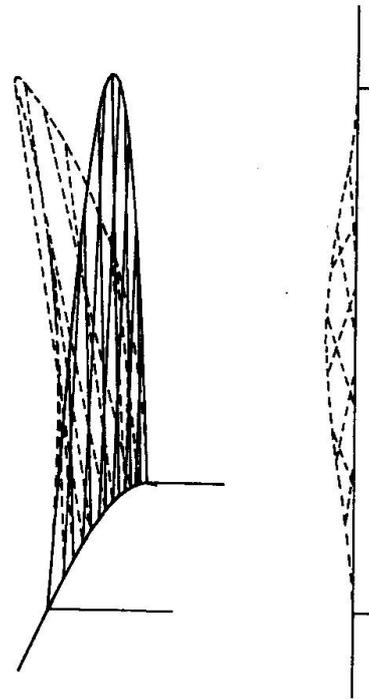Fig. 5 Single arch idealization

Fig. 6 First instability mode
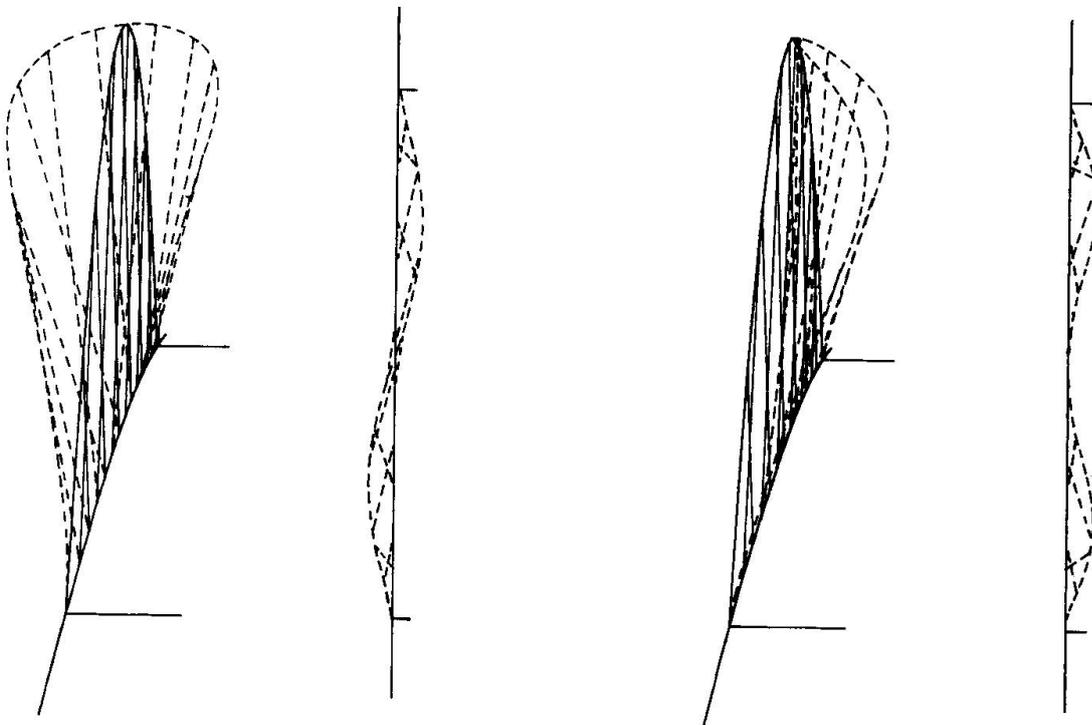   without a crown support
      $\lambda_1 = 2.0001$

Fig. 7 Instability modes 1 and 2 with crown support.
      $\lambda_1 = 2.04 - \lambda_2 = 4.211$

4. STABILITY COMPUTATION FOR THE WHOLE STRUCTURE.

To clear up this problem of the crown rotational restraint, a three-dimensional analysis of the whole final design was performed by means of truss bars finite elements (for the hangers) and beam elements (for the arches, the girders, the crown bracing strut and the floor beams including a contributing width of the reinforced concrete deck): fig. 8.

For this idealization again, only the instability modes were computed. The first two are illustrated on a plan view in fig. 9. They exhibit an obvious similarity with modes 1 and 2 in the case of a single arch restrained by a simple support at its crown. But, because of the crown bracing strut, the symmetry of displacements about the central point of the bridge entails longitudinal displacements of both arches at their crown against which they oppose a strong stiffness: the critical load multiplier is raised from $\lambda = 2,04$ to $\lambda = 2,60$; this emphasizes the importance of a *correct simulation of the elastic support provided by the crown bracing strut*.

An alternative design with two bracing struts instead of a single one, located at third span of the arches, brought an additional benefit of 14 % for the same first instability mode but was discarded for technological reasons.
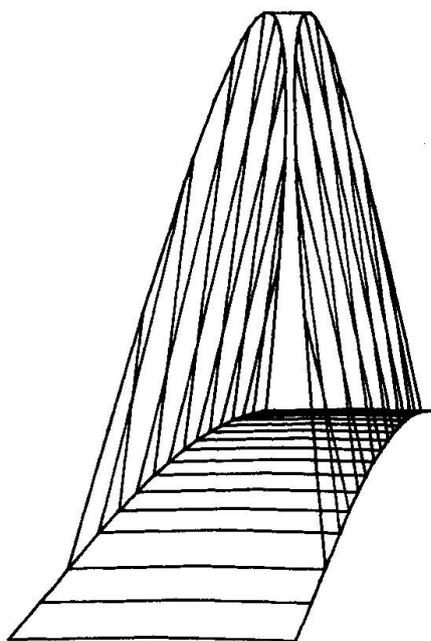

Fig.8 Idealization of the whole structure
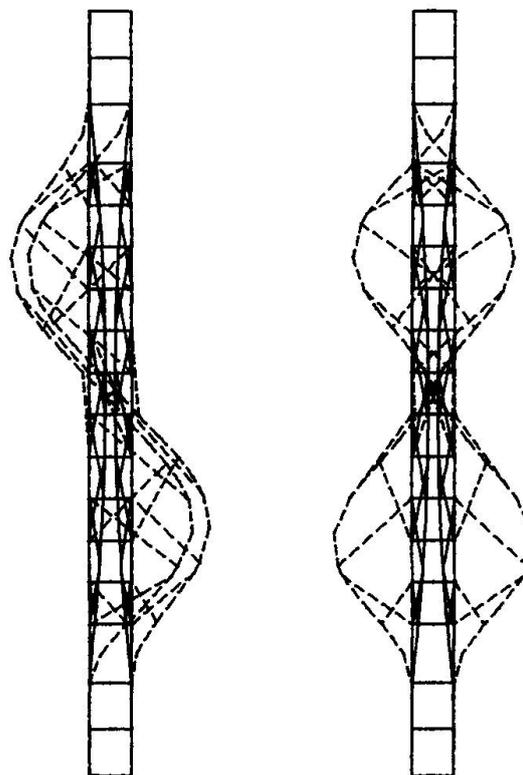
Fig. 9 Instability modes 1 and 2.
$\lambda_1 = 2.665$ - $\lambda_2 = 3.465$

5. NEW STEP-BY-STEP COMPUTATION OF AN ARCH IDEALIZED AS A STRAIGHT BEAM.

A step-by-step computation of the whole structure was possible albeit costly for the whole tridimensional structure. The confidence gained with the preliminary design computation led the consulting office to prefer a step-by-step analysis of a new fictitious straight beam. At midspan however the bracing strut is simulated by a simple support preventing out-of-plane displacements and an elastic spring restraining the rotation about a vertical axis. This rotational spring was given a stiffness of 5000 T x m/radian, value arrived at in two

different ways giving identical results :
- by correctly estimating the rotational stiffness at one end of the crown bracing strut ;
- by adjusting this rotational stiffness parameter until the same critical load factor λ be obtained for the fictitious straight beam as for the previously computed three-dimensional structure.

The results of the step-by-step computation possess the same general outlook as those already shown in fig. 4 ; given the same loads however the displacements are smaller and the instability asymptotic line is raised : this is understandable since the stiffness of the crown bracing strut is better taken into account.

This step-by-step computation is useful to check the magnitude of the second order out-of-plane displacements under service loads. It also gives the axial stresses and second-order out-of-plane bending moments in the arch. These beam forces are in turn used to check that the resulting stresses in the arch box-girder are acceptable and to justify the fact that the elasto-plastic part of the computer program was inactivated.

6. ADDITIONAL COMPUTATIONS.

Local buckling may occur in the arch box-girders, namely in the flange plates which are axially compressed along their direction of curvature. Use was made of the results of a previous study performed by finite elements on compression curved plates [7].

Vibration modes and eigenfrequencies were also computed for the three-dimensional model of fig. 8. As expected, similar shapes are found among the vibration and instability modes : modes 1 and 2 in particular exhibit nearly identical shapes (fig. 9).

7. CONCLUSIONS.

Much simplified finite elements models, namely fictitious plane beams idealizations, were used to analyze the three-dimensional stability of a bowstring bridge of unusual design. Research of instability modes and step-by-step analyses, in the case of the fictitious beams, single arches and three-dimensional structures, including tentative computations, took less than 1850 seconds of CPU time on an IBM 370-158 Machine.

The bridge is now being completed (fig. 10). To avoid interference with the traffic on the King Albert Canal, the bridge was erected on the bank, shifted on barges and floated into place during the summer of 1982.

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Fig. 10 Bridge at Hermalle.