PB95-149696

NTIS
Information is our business.

DEVELOPMENT OF WOOD BRIDGES: SCALE MODEL TESTS OF A WOODEN ARCH BRIDGE

HELSINKI UNIV. OF TECHNOLOGY, ESPOO (FINLAND)

1994

U.S. DEPARTMENT OF COMMERCE
National Technical Information Service

BIBLIOGRAPHIC INFORMATION

PB95-149696

Report Nos:

Title: Development of Wood Bridges: Scale Model Tests of a Wooden Arch Bridge.

Date: 1994

Authors: H. Rautakorpi, A. Jutila, R. Maekipuro, P. Haakana, and L. Salokangas.

Performing Organization: Helsinki Univ. of Technology, Espoo (Finland). Lab. of Bridge Technology.

Type of Report and Period Covered: Research rept.

Supplemental Notes: Also pub. as Helsinki Univ. of Technology, Espoo (Finland). Lab. of Bridge Technology rept. no. PUB-10. See also PB95-149688 and PB95-149704.

NTIS Field/Group Codes: 50A (Highway Engineering)

Price: PC A03/MF A01

Availability: Available from the National Technical Information Service, Springfield, VA. 22161

Number of Pages: 28p

Keywords: *Model tests, *Scale models, *Design analysis, *Bridge design, *Structural analysis, Joints(Junctions), Displacement, Deflection, Dimensions, Beams(Structural), Test facilities, Structural design, *Foreign technology, *Wooden arch bridges.

Abstract: The report is concentrating on a scale model of a wooden arch bridge. The span of the model, which is thought to represent a real bridge in scale one to ten, is 4.2 meters. Externally the model is supported as a simple beam, which means that no external redundant force for the arch is provided. The report includes overall sketches as well as detail drawings of the most important parts of the model. The loading cases considered are the dead load, the wind load on the arches, point loads on the bridge deck and a distributed load on one half of the span. In the last case mentioned the load was gradually increased almost until the rupture load. The loading arrangements are introduced by drawings and photographs. During loading deflections and horizontal displacements were measured at several points around the model. The most important test results are presented graphically. According to the tests the horizontal rigidity of the model seems to be less than assumed by the theory used. The reason is found by the loosened joints of the bracing members. On the other hand, the deck plate makes the model quite rigid for vertical loads.

PB95-149696

HELSINKI UNIVERSITY OF TECHNOLOGY
Department of Structural Engineering
Laboratory of Bridge Engineering

Publication No. 10

Heikki Rautakorpi
Aarne Jutila
Risto Mäkipuro
Pekka Haakana
Lauri Salokangas

Research project

DEVELOPMENT OF WOOD BRIDGES

Scale model tests of a wooden arch bridge

Otaniemi 1994

REPRODUCED BY: NTIS
U.S. Department of Commerce
National Technical Information Service
Springfield, Virginia 22161

PB95-149696

ABSTRACT

This report is concentrating on a scale model of a wooden arch bridge. The span of the model, which is thought to represent a real bridge in scale one to ten, is 4.2 metres. Externally the model is supported as a simple beam, which means that no external redundant force for the arch is provided. The model is designed to correspond with a real structure loaded by normal pedestrian load of 4 kN/m^2 or a maintenance vehicle load of 120 kN.

The report includes overall sketches as well as detail drawings of the most important parts of the model. The loading cases considered are the dead load, the wind load on the arches, point loads on the bridge deck and a distributed load on one half of the span. In the last case mentioned the load was gradually increased almost until the rupture load. The loading arrangements are introduced by drawings and photographs.

During loading deflections and horizontal displacements were measured at several points around the model. The most important test results are presented graphically. According to the tests the horizontal rigidity of the model seems to be less than assumed by the theory used. The reason is found by the loosened joints of the bracing members. On the other hand, the deck plate makes the model quite rigid for vertical loads.

ACKNOWLEDGEMENTS

This report was prepared in the Laboratory of Bridge Engineering, Helsinki University of Technology. Financial support was given by the Finnish financial organization TEKES (Technology Development Centre), by the Finnish National Road Administration and by the industrial plant for wood working, Vierumäen Teollisuus Ltd.

The report is part of the research program Development of Wood Bridges. The leader of the research project is Professor Aarne Jutila. The other members of the leader group are:

Mr. Matti Kuusivaara, M. Sc., from the Finnish National Road Administration, Chairman,
Mr. Keijo Kopra, Managing Director, from Vierumäen Teollisuus Ltd,
Professor Alpo Ranta-Maunus, from the Technical Research Centre of Finland and
Mr. Olli-Pekka Nordlund, M. Sc., from TEKES.

The authors wish to express their gratitude to the persons and organisations mentioned above and to all those who contributed to this study.

TABLE OF CONTENTS

1. INTRODUCTION

This investigation belongs to a wider project of wood bridges started in Helsinki University of Technology during the year 1992. From the very beginning of the project it was found that an arch is a proper type of bridge especially for light traffic. Therefore, it was decided to design a scale model of such a bridge and to investigate its function under various loads. This is expected to give information for the design of such bridges and for the validity of different assumptions made during the design.

2. GENERAL FEATURES OF SCALE MODELS

A scale model is a reduced model of a real structure. The ratio of dimensions between the real structure and the model is generally an integer denoted here by n, i.e., the scale is 1:n. It is important to find proper scale factors between the quantities of the model and the corresponding quantities of the real structure. It can be shown that the relations of Table 1 are valid for different quantities.

Table 1. Equivalence of quantities between a real structure and the corresponding scale model.

Parameter	Unit	Real structure	Scale model
Length	m	l	l / n
Concentrated load	kN	F	F / n^2
Distributed load	kN/m^2	p	p
Mass	kg/m^2	m	m / n
Displacement	m	w	w / n
Natural frequency	Hz	f	n f
Stress	MN/m^2	σ	σ

Even if a scale model is made of the same material as the real structure, its self weight does not follow the same law as the distributed load presented in Table 1. Therefore, an additional surface load

$$g(1-\frac{1}{n})$$

is needed to compensate the lack of the dead load of the model.

3. MODEL GEOMETRY AND MATERIALS

To check the validity of theoretical calculations, a scale model of a wooden arch bridge was constructed. It was designed to represent a real bridge loaded by light traffic as follows:

- uniformly distributed load p = 4 kN/m^2 and
- a maintenance vehicle load F = 120 kN according to the scheme of Fig. 1.

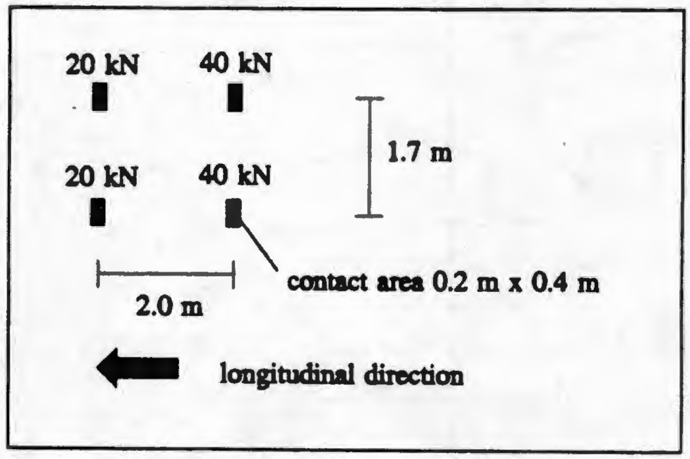

Fig. 1. Maintenance vehicle load arrangement of the light traffic bridge used as the base of the scale model studied.

A general view of the model is presented in Fig. 2 and the details of the structure in Figures 3 to 7. The model can be assumed to represent a real arch bridge with a span of 42 metres and a nominal width of 4.5 metres, i.e., the scale factor is n=10.

The type of bridge is a two-hinge tied arch, where the horizontal component of the arch force is taken by tension bars at the road level so that the external support reactions are vertical. On each side of the bridge deck there are double arches and a tension bar between the arches (cf. Fig. 3). A transversal bracing truss joins the double arches to each other. The truss is composed of wooden verticals and steel diagonals to assure the horizontal stability against buckling and wind forces. The bridge deck is composed of crossbeams and a massive wooden deck plate on the beams. It is suspended from the arches with steel hangers.

The main material of the scale model was natural wood of Finnish pine. The arches and the tension bars were composed of five laminations glued together with melamine glue. Other wooden components were solid wood. The mechanical properties of the glulam were determined with bending tests, which gave the average modulus of elasticity of

12400 MN/m^2. The average dry density and moisture content were measured to be 490 kg/m^3 and 0.06, respectively.

In the vicinity of the supports, the double arches were reinforced and joined by thin plywood plates to form a box-type structure (Fig. 4). The hangers and diagonals of the bracing truss were threaded rods of stainless steel with the ultimate strength of 520 MN/m^2. The joints (Figures 5, 6 and 7) were made with screws with the ultimate strength of 800 MN/m^2 and steel plates (Figures 6 and 7) with the ultimate strength of 520 MN/m^2. The number of screws was chosen so that the maximum shear forces under the equivalent loads were approximately the allowable ones.

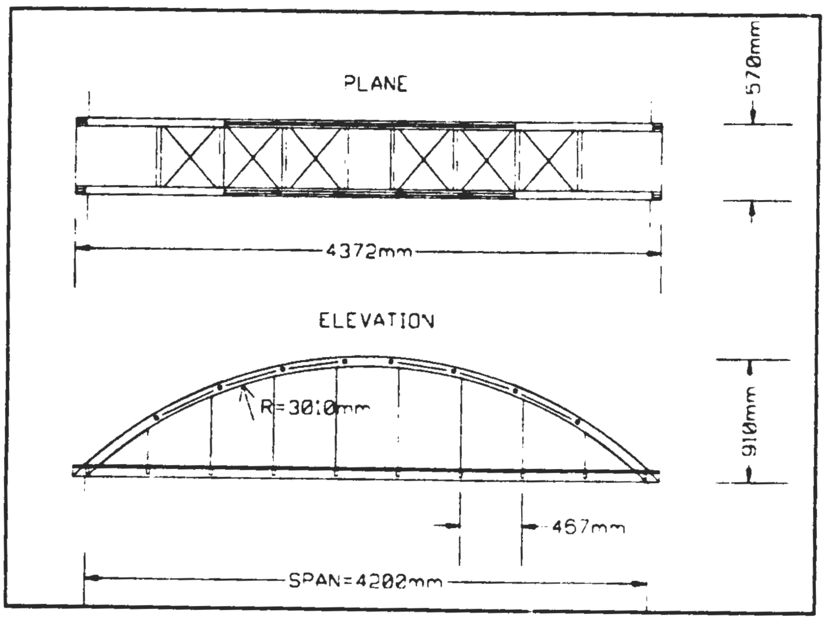

Fig. 2. General view of the arch model.

The deck plate of the model was made of birch plywood with nine veneers, the grain direction of face veneers being parallel to the span of the arch. The plate was fixed on each crossbeam with five nails whose length was 30 mm and thickness 1.7 mm. The deck plate was not continuous, but it was made of three parts so that there was a transverse joint at each third crossbeam.

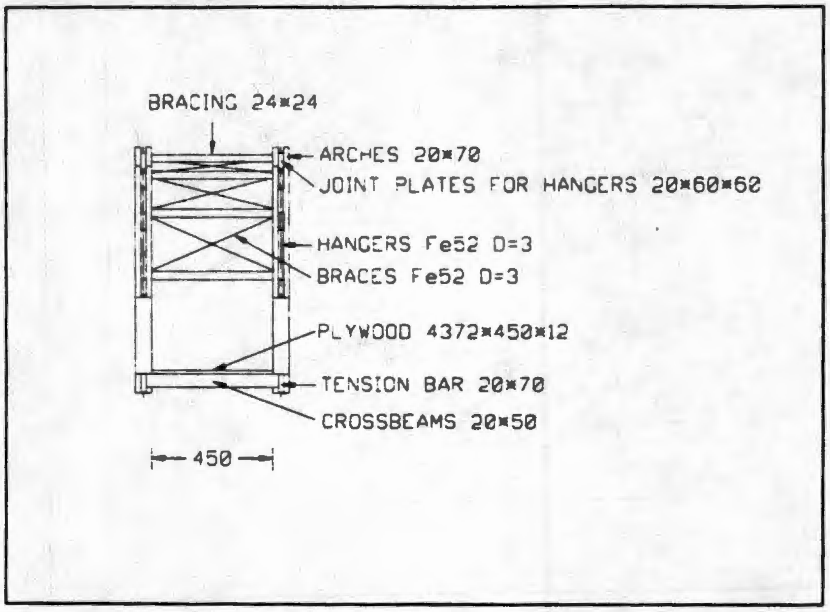

Fig. 3. Cross-section of the arch model. Dimensions in millimetres.

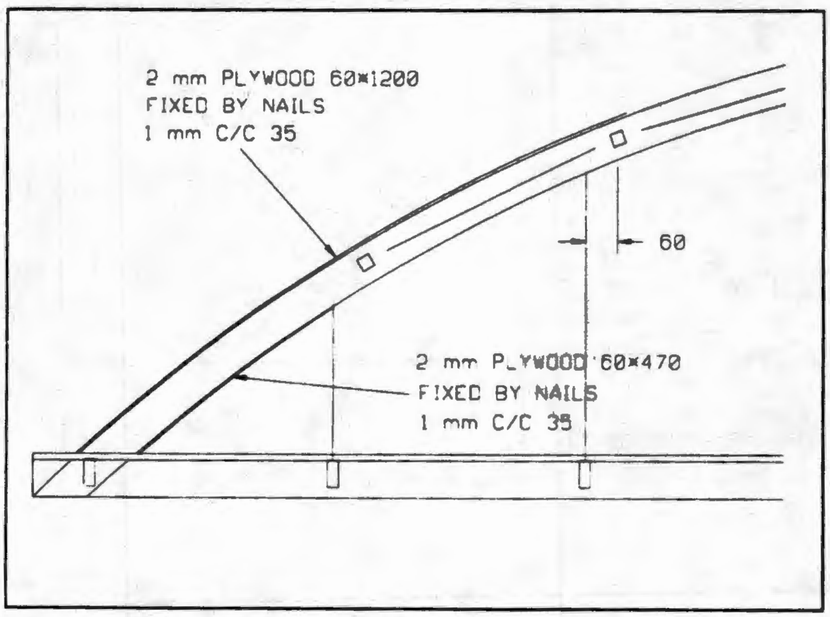

Fig. 4. Detail of the arch footing. Dimensions in millimetres.

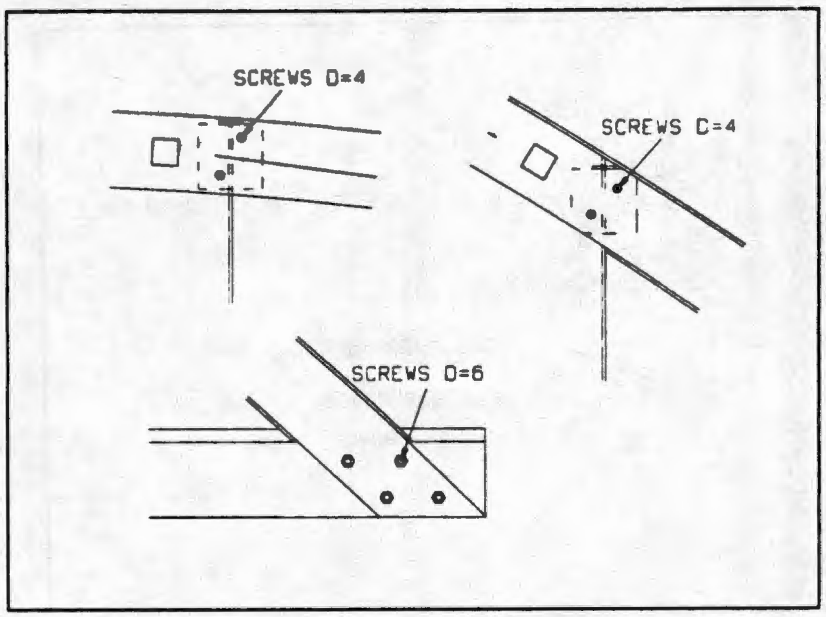

Fig. 5. Joint details of the arch model. Dimensions in millimetres.

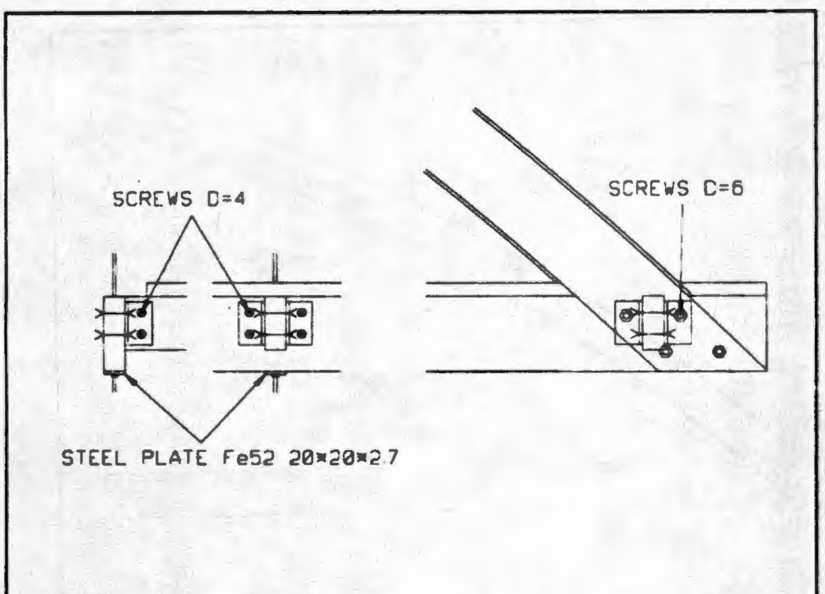

Fig. 6. Connections between the crossbeams and the tension bar. Dimensions in millimetres.

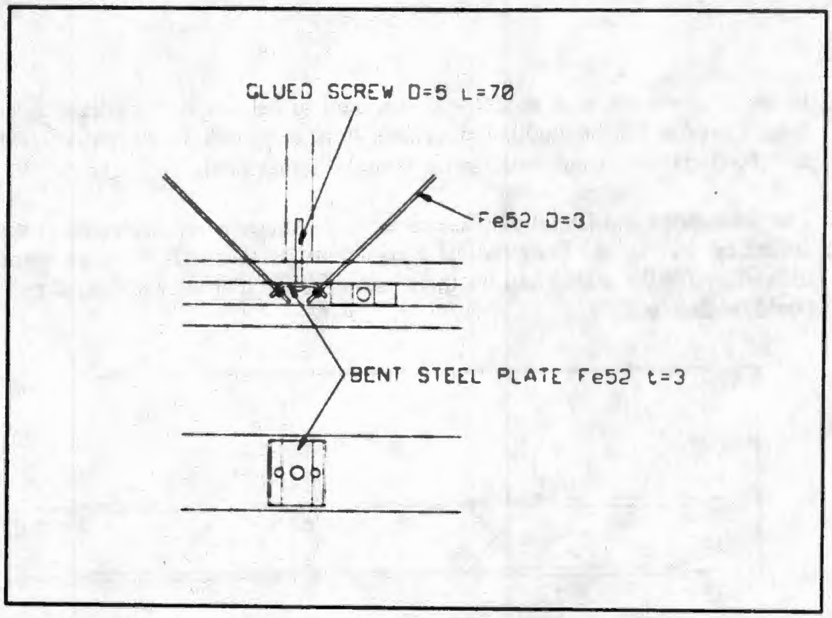

Fig. 7. A joint between braces and the arch. Dimensions in millimetres.

4. TEST ARRANGEMENTS

In the analysis the arch model was assumed to be simply supported at one end and hinged but free for longitudinal movement at the other end. In the transverse direction the arch footings were supported against lateral displacements.

The deflections and lateral displacements of the model were measured at several points according to Fig. 8. Longitudinal movements at the arch footings were registered differently for the arches and for the tension ties, so that the eventual slips of the joints could be detected.

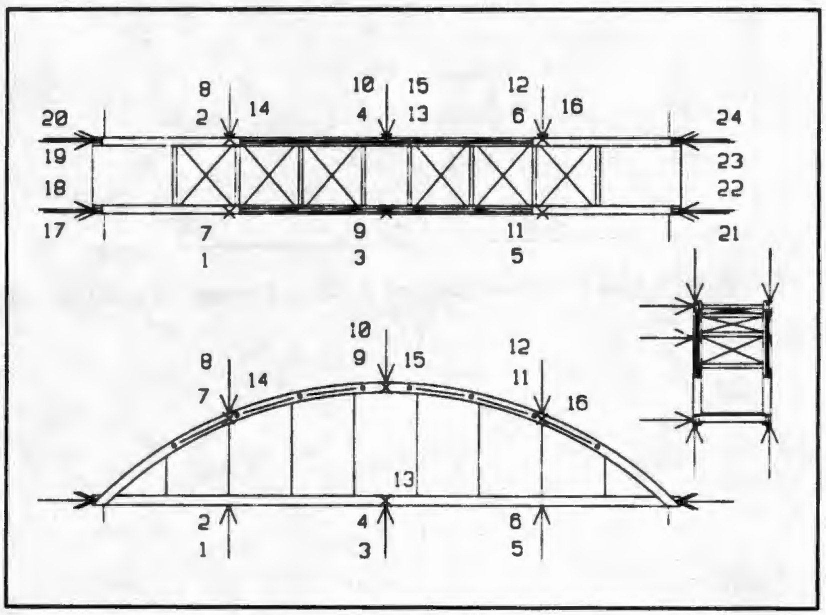

Fig. 8. The measuring points and directions of displacements used in the scale model testing.

The main design loads of a light traffic arch bridge are as follows:
- the dead weight of the bridge,
- a uniform pedestrian load distributed on a certain area of the bridge deck,
- a maintenance vehicle load at an arbitrary point of the deck and
- the wind load in the transverse direction.

All these alternatives were considered when choosing the loading cases of the scale model.

The dead load and the concentrated loads were produced by steel plates as shown in Fig. 9. The wind load was approximated with the aid of four lateral point loads according to Fig. 10. The horizontal point loads were caused by weights fixed to the ends of steel wires bent over a horizontal steel cylinder. Friction between the wire and the cylinder was measured and found to be so large that the horizontal force was only about 60% of the vertical load caused by the weights.

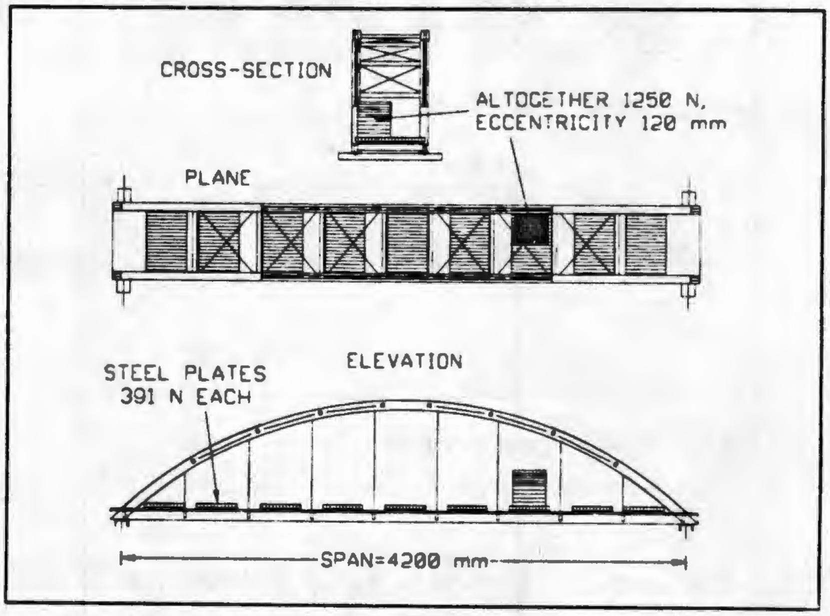

Fig. 9. Scale model loading corresponding to a maintenance vehicle load.

The loading sequence of the dead load, the wind load and the concentrated loads is presented in Figures 11 and 12. The test was started by a loading case corresponding to the dead load of the real bridge. This load was affecting the structure during a weekend so that information about the creep properties of the wood was obtained. In the case of a vehicle load, a concentrated load was positioned at the centre span or at about the quarter span of the arch. In both cases the load was positioned eccentrically with respect to the centre line of the cross-section. The concentrated load was placed by turns at equivalent points, two points at the centre span (Fig. 11) and four points at the quarter spans (Fig. 12). By this method several test results were obtained for one test specimen.

The distributed load on one half of the arch only was simulated by four point loads as shown in Fig. 13. The load denoted by Q in the figure was caused by a steel wire pulled by a tackle and an electric motor. The wire was led through a hole in the deck and attached to a load indicator resting on a load distributing beam. The loading rate was chosen so that the maximum deflection of 0.5 millimetres was reached within a minute.

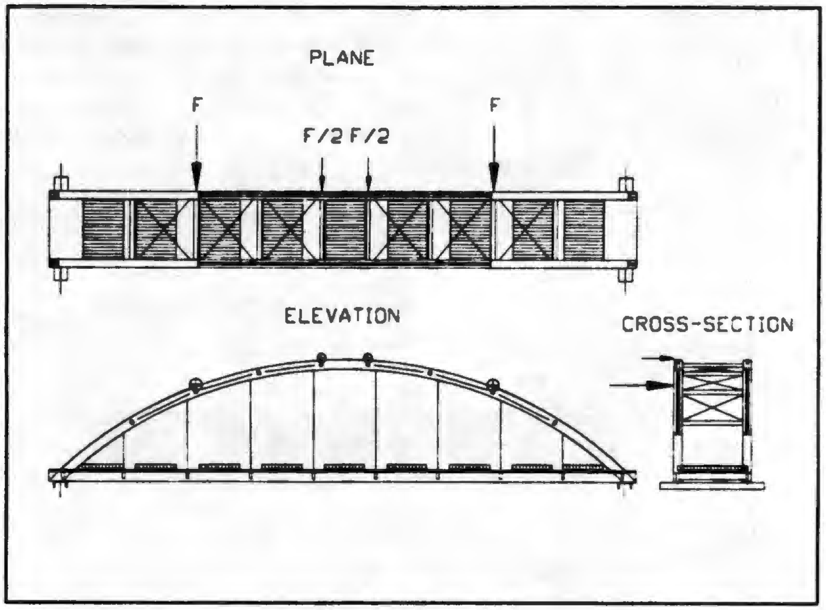

Fig. 10. **Wind load approximation by four point loads.**

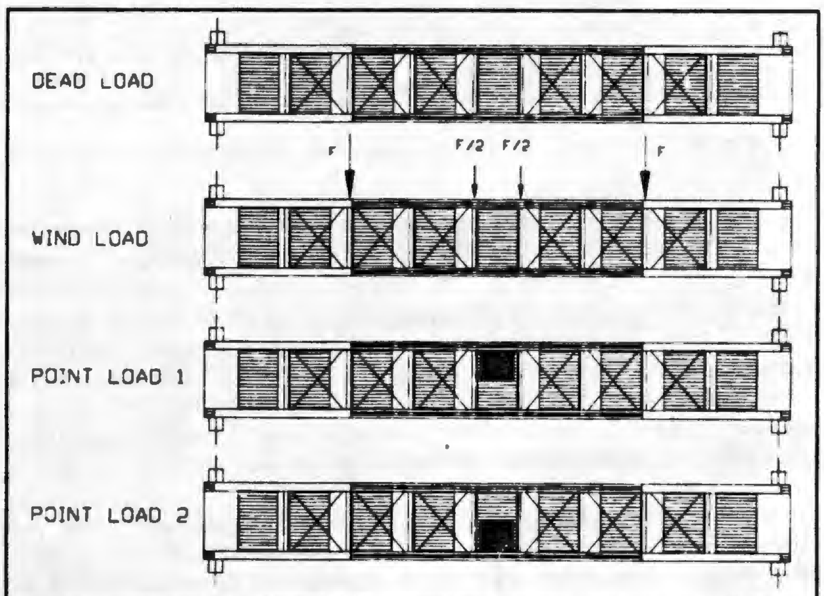

Fig. 11. **Load arrangements for dead load, wind load and concentrated loads at the centre of the span.**

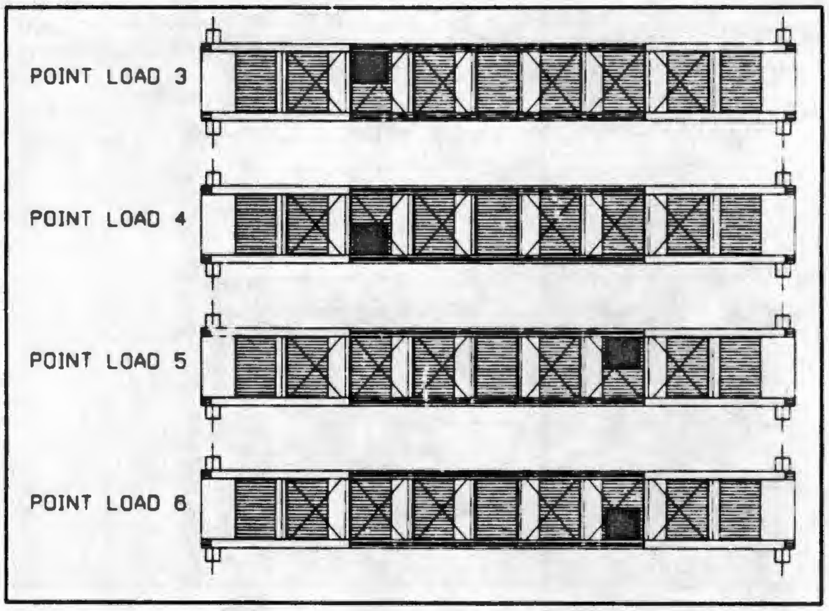

Fig. 12. **Test arrangements for the asymmetric concentrated loads.**

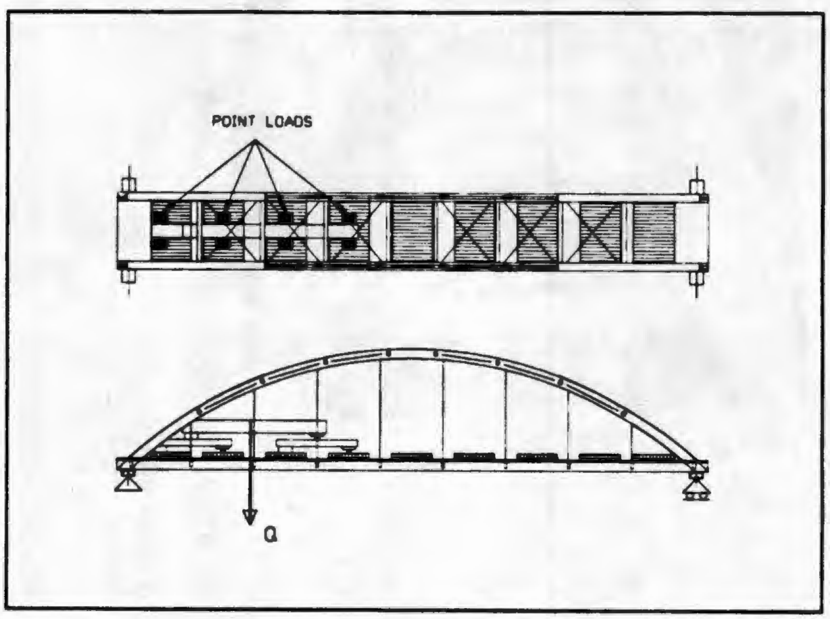

Fig. 13. **Test arrangements for the distributed load.**

The original intention was to load the scale model up to rupture with the increasing distributed load but, due to the higher ultimate loads than expected, the capacity of the loading equipment was reached before the required force. Thus, the test was interrupted at a level of loading corresponding to a fivefold pedestrian load (20 kN/m^2). The total value of the distributed load was then 17.4 kN.

Figures 14 to 17 present some photographs of the scale model and the test arrangements.

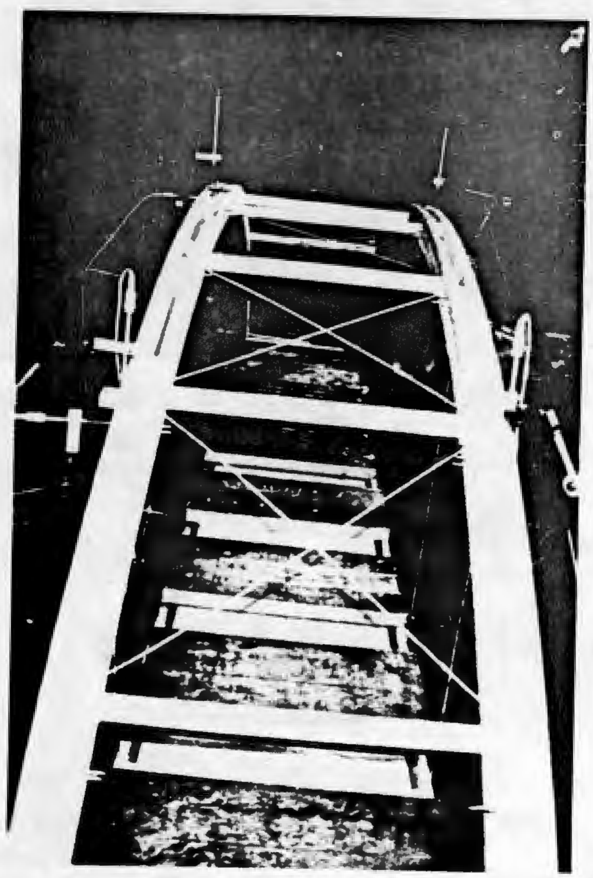

Fig. 14. A photograph of the scale model and test arrangements.

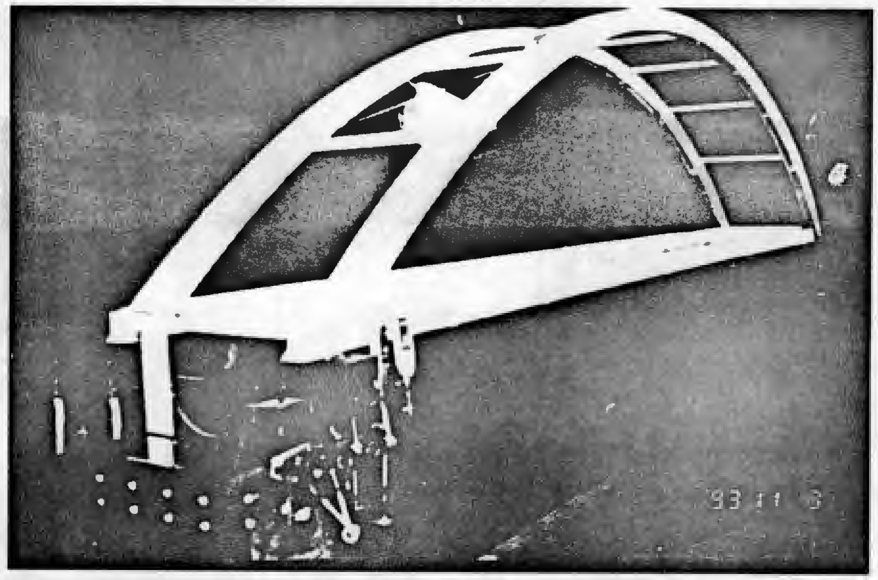

Fig. 15. A general view of the scale model.

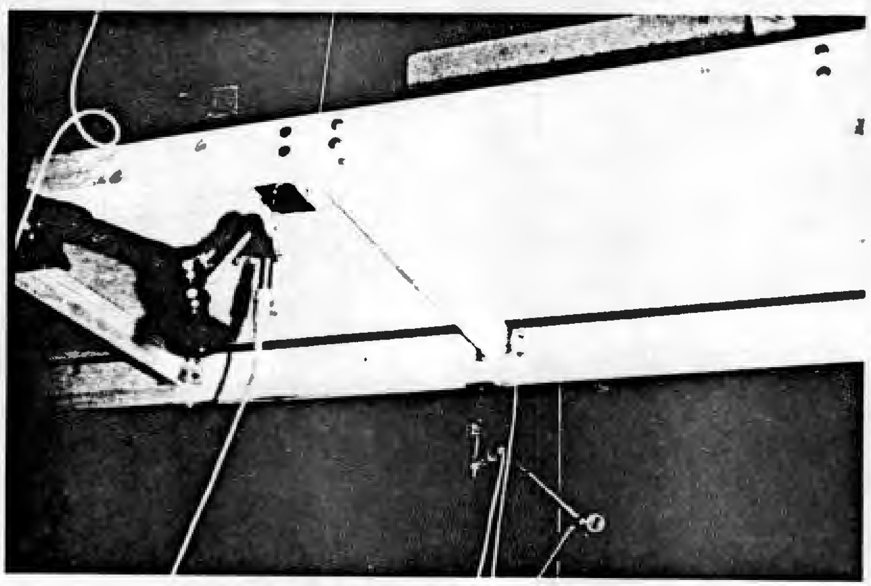

Fig. 16. A view of the test arrangements beneath the bridge deck.

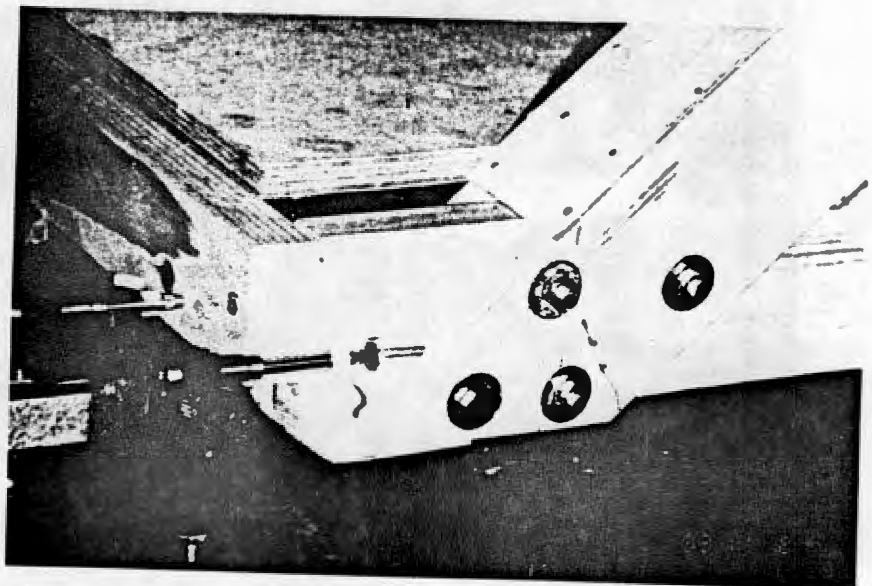

Fig. 17. A detail photo of the joint at the arch footing.

5. TEST RESULTS WITH ANALYSIS

All test results were compared with corresponding values obtained from computer calculations based on linear elastic plane frame and grillage programs. Generally, the deck plate was not considered in the calculations. The diameter of the steel hangers was approximated to be 3 mm. The modulus of elasticity of steel was assumed to be 210000 MN/m^2. In torsion, the modulus of rigidity of wood was assumed to be 810 MN/m^2.

At first, the model was loaded by dead load according to Fig. 11 and the deflections were measured and compared with calculated values. The measured deflections were found to be about 40% larger than the calculated ones. This is due to the fact that at the beginning the joints are not tight enough to prevent a slip. The model must be slightly loaded before it begins to behave linearly as assumed in the calculations.

Fig. 18 shows the results of the creep test. The curves represent the measured relative deflection as a function of time at points no. 3, 4, 9 and 10 of Fig. 8, respectively. It is seen that within 60 hours the additional deflection due to creep was about 20% with a variation of about ± 4%.

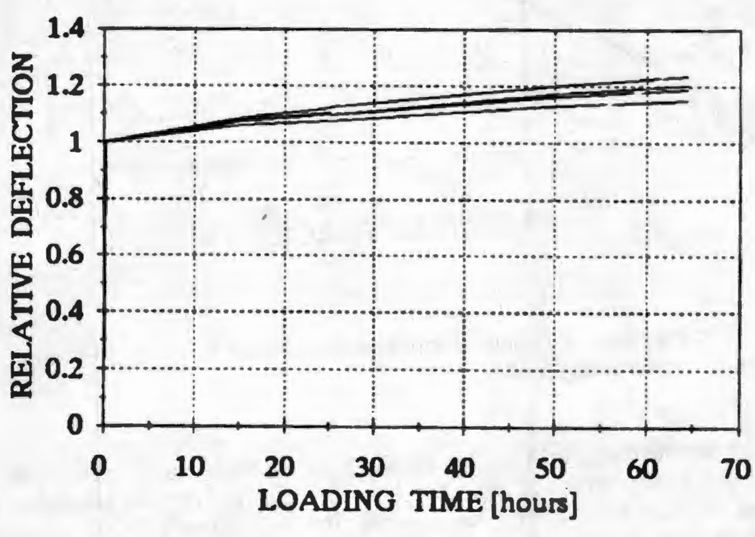

Fig. 18. The measured deflection increments of the model at points no. 3, 4, 9 and 10, respectively, when loaded by the dead load.

The results of the transverse loading shown by Fig. 10 are presented in Fig. 19. When loading, the lateral forces F were increased in stages to the value of F=93 N and then removed. It can be seen that the two curves representing loading and unloading of the scale model are about the same with the exception at zero load. This means that a permanent displacement of 2.6 mm remained when the load was removed.

As shown, the load-displacement curve for the lateral loads is not linear especially when unloading. One reason for this may be that the joints between the braces and the arch, which could not be tightened afterwards, were loosened because of drying. Nevertheless, an experimental value for the displacement was determined with a regression line for the three largest force values shown by the dark line in Fig. 19.

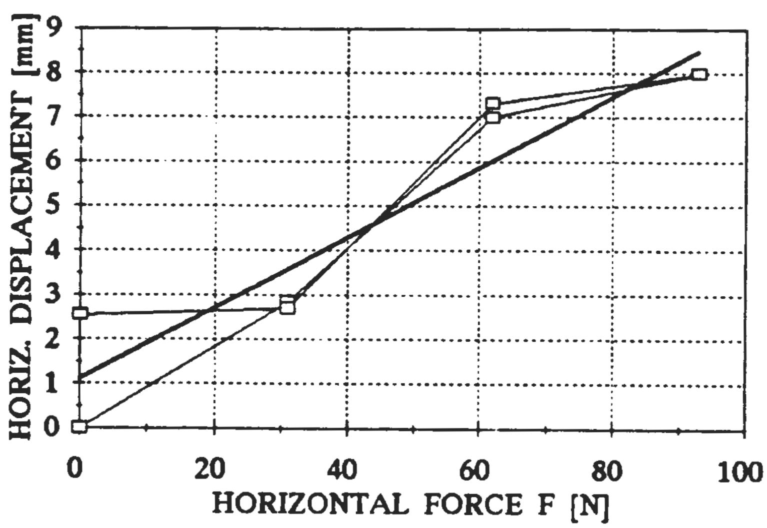

Fig. 19. The measured horizontal displacement at the crown (point 15 in Fig. 8) and the corresponding regression line.

Fig. 20 shows experimental results for the distortion of the bridge cross-sections in the middle and at the quarter point of the span. It is seen that at both cross-sections the distorted shape reminds each other. Consequently, the largest rate of distortion takes place near the ends of the arch. In accordance with the regression line of Fig. 19, the maximum lateral displacement is 7.9 millimetres. The calculated displacement was 1.9 mm, which is only 24% of the corresponding experimental value. Therefore, the model was recalculated as a frame structure neglecting diagonal braces and the displacement of 17.1 mm was obtained. A comparison of the results leads to the conclusion that only about 60% of the braces were acting during testing.

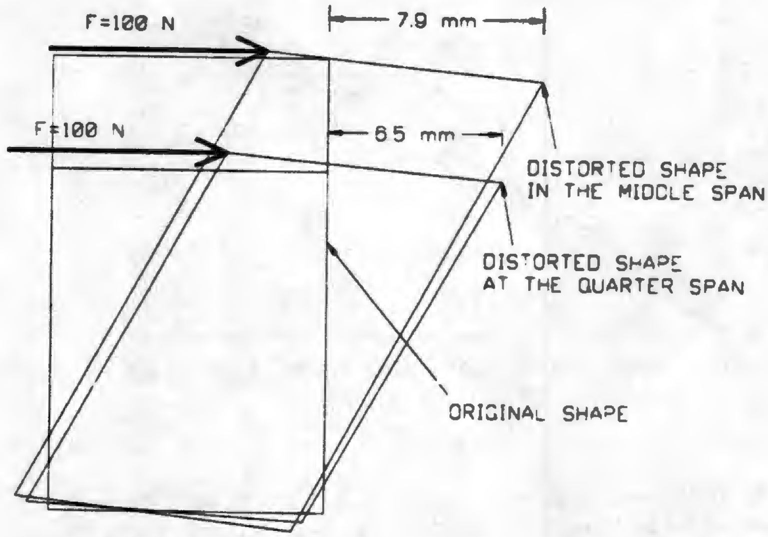

Fig. 20. Distortion of the cross-section of the scale model in the middle and at the quarter span, respectively, due to the wind load.

Fig. 21 shows the load-deflection curves corresponding to loading cases shown in Fig. 12. The deflection was measured at points 2, 1, 6 and 5 of Fig. 8, respectively. Only three of the test results are shown in Fig. 21, because one of the deflection indicators did not function well. One can see that the curves are approximately linear but in all cases there is a tiny permanent deflection. The calculated deflection due to the point load was obtained by using a plane frame model, which gave the maximum deflection of 4.2 mm when the load was 1200 N. Thus, the experimental value was only 81% of the corresponding theoretical one.

In the case of point loads 1 and 2 shown in Fig. 11, the maximum experimental deflection was about 89% of the calculated one. The differences between the experimental values and corresponding calculated ones are caused by the fact that in reality the deck plate makes the model more rigid and thus decreases the deflection.

Two measured deflection curves for the loading case shown in Fig. 13 are presented in Fig. 22. The deflection was measured at points 1 and 2 shown in Fig. 8. It is seen that the model behaves approximately linearly until the registered maximum load.

The experimental and calculated deflections are compared in Table 2 and in Fig. 23. For the calculated results two different methods were used: in one method the bending as well as the tensile stiffness of the deck plate were left without any consideration. In the other method they were assumed to be fully active. The results show that the experimental values lie between these two limits, which states that the model behaves according to the theory used.

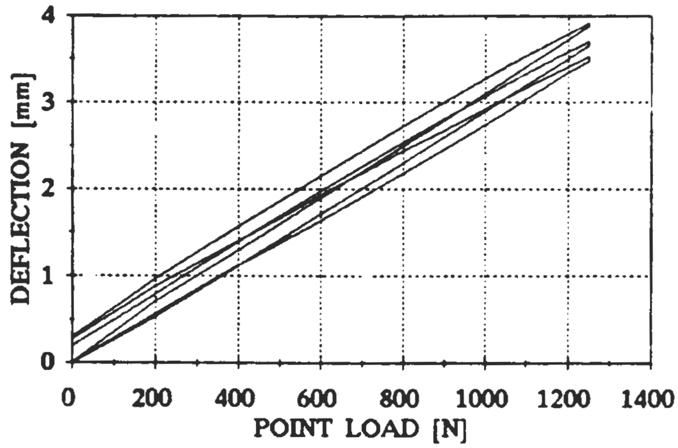

Fig. 21. Deflections measured at points no. 2, 1 and 6, when the model was loaded by point loads 3, 4 and 5 of Fig. 12, respectively.

During the tests the lateral displacements of the arch were also measured. In the case of the distributed load of Fig. 13, the maximum value was about 2 millimetres, which is a surprisingly low value considering the poor lateral rigidity of the model.

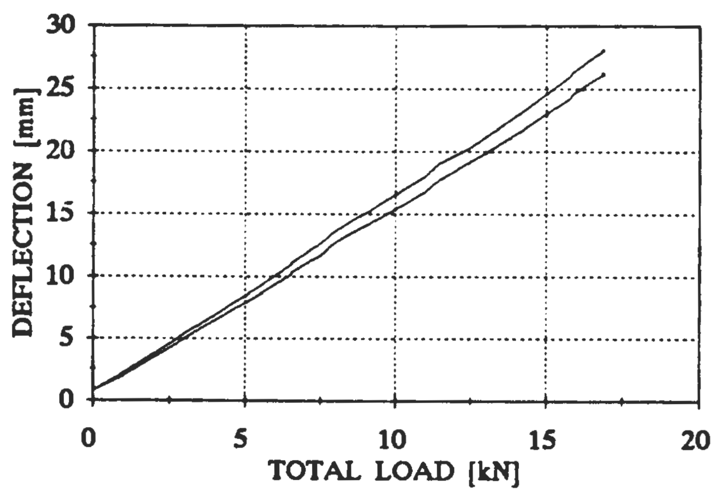

Fig. 22. Deflections measured at points no. 1 and 2 of Fig. 8, respectively, as a function of the total load shown in Fig. 13.

Table 2. Measured and calculated deflections of the arch model when loaded by a distributed load presented in Fig. 13.

Method used		Deflection [mm] due to total load of 1 kN at the points specified below					
		1 and 2	3 and 4	5 and 6	7 and 8	9 and 10	11 and 12
Ana-lyti-cal	Without considering the effect of the deck plate	1.76	0.06	-1.52	1.71	0.05	-1.53
	With considering the effect of the deck plate	0.93	0.06	-0.78	0.90	0.04	-0.79
Experimental		1.55	0.21	-1.11	1.35	0.18	-1.14

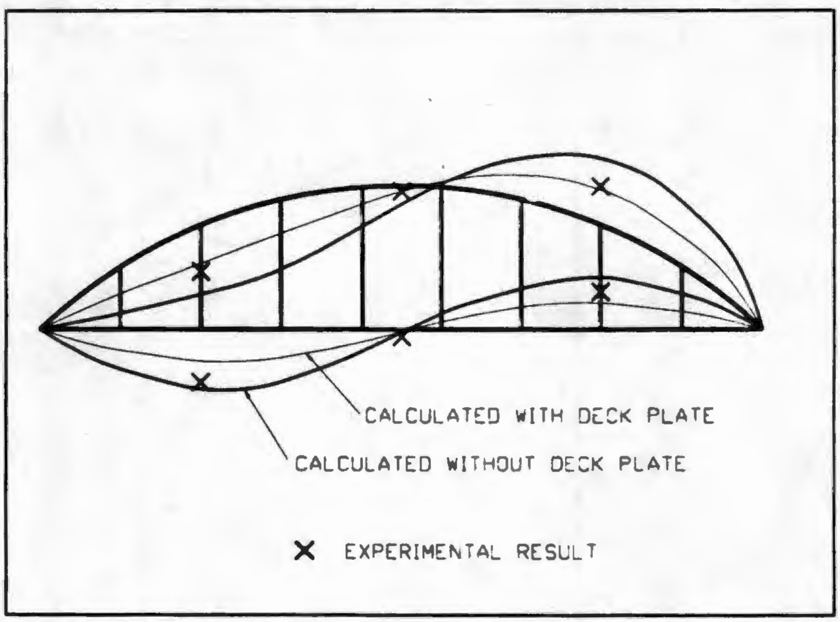

Fig. 23. Comparison between the calculated and experimental deflections due to the loading case shown in Fig. 13.

6. CONCLUSIONS

The main conclusions from the scale model tests of the arch bridge are as follows:

- The bridge deck functions as part of the main load carrying system. This property should be exploited when designing real tied arch bridges. The role of the deck can be improved further by increasing the lateral stiffness of the joints.

- There is a risk that moisture changes loosen the joints of the bracing elements and due to this the lateral rigidity of an arch bridge may remain smaller than expected by calculations.

As a final general conclusion it can be stated that for vertical loads the tested scale model proved to act as expected by the linear plane frame theory used in the investigation. For stability forces and wind loads, an arch bridge can be approximated as a transversal plane frame that contains the arches, the crossbeams and the diagonals. Diagonals do not fully act, but no exact rules for their consideration can be given on the basis of this study. This question would require further investigations.

REFERENCES

[1] Puurakenteiden suunnitteluohjeet (Design rules for wood structures). RIL 120-1991. Suomen Rakennusinsinöörien Liitto RIL r.y., Helsinki. 147 p.

[2] Rautakorpi, H., Tesár, A., Jutila, A., Mäkipuro, R., Haakana, P. & Salokangas, L., Development of wood bridges, Prospects of wood in various types of bridges. Helsinki University of Technology, Laboratory of Bridge Engineering, Publication No. 5. Otaniemi 1993. 54 p.

[3] Siltojen kuormat (Loads for bridges). Tielaitos, Tiehallitus. Helsinki 1991. 28 p.

Publication serie of the Laboratory of Bridge Engineering

1. Tesár, A., *Advanced analysis of thin-walled structures in bridge and structural engineering.* Editor Jutila, A. 1992. 82 p.

2. Haakana, P., Jutila, A., Rantakorpi, H., Salokangas, L., *Tutkimusprojekti puusiltojen kehittäminen, taustatietojen selvittäminen.* 1993. 78 p.

3. Haakana, P., Jutila, A., Rantakorpi, H., Salokangas, L., *Research project development of wood bridges, survey of source documents.* Excerpt of the Finnish Report *Survey of background information* (Publication no. 2). 1993. 46 p.

4. Kiviluoma, R., *[illegible] analyysi [illegible].* 1993. 112 p.

5. Rantakorpi, H., Tesár, A., Jutila, A., Mäkipuro, R., Haakana, P., Salokangas, L., *Research project development of wood bridges, prospects of wood in various types of bridges.* 1993. 54 p.

6. Tesár, A., Jutila, A., Rantakorpi, H., Haakana, P., Salokangas, L., *Research project development of wood bridges, numerical and experimental analysis of static and dynamic behaviour of wood bridges.* 1993. 47 p.

7. Wiio, M., Jutila, A., Mäkipuro, R., Haakana, P., Salokangas, L., Wiio, J., *Research project development of wood bridges, literature survey of shear connections of wood-concrete composite bridges.* 1994. 14 p.

8. Rantakorpi, H., Jutila, A., Mäkipuro, R., Haakana, P., Salokangas, L., *Research project development of wood bridges, literature survey of wooden arch bridges.* 1994. 34 p.

9. Haakana, P., Jutila, A., Kiviluoma, R., Mäkipuro, R., Rantakorpi, H., Salokangas, L., *Tutkimusprojekti puusiltojen kehittäminen, [illegible] [illegible] ja [illegible].* 1994. 66 p.

10. Rantakorpi, H., Jutila, A., Mäkipuro, R., Haakana, P., Salokangas, L., *Research project development of wood bridges, scale model tests of a wooden arch bridge.* 1994. 26 p.

END
FILMED

DATE:
3-3-95

NTIS