TESTS ON THE STEEL STRUCTURE OF A RAILWAY BRIDGE

By

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I. Introduction

The reconstructed railway bridge system across the Tisza River at Algyő, inaugurated December 9th, 1976, is composed, as a matter of fact, of three independent steel truss bridges (Fig. 1).

In 1960, the original single-span structure in the middle had been replaced by a simply supported riveted truss bridge with secondary diagonals and reinforced concrete deck plate. In 1976, two welded HT (high-tensile prestressed) bolted trusses with continuous main girders and orthotropic deck plate were erected in the flood plain, designed by UVATERV (Road and Railway Design Office). The steel structure was manufactured and erected by the Bridge Factory of Ganz-MÁVAG Co.

The Department of Steel Structures, Technical University, Budapest was commissioned to test the spans VI to VIII over the flood plain (Fig. 2), to be reported below.

2. Test circumstances

Aspects in establishing the test program were:

— Up-to-date railway truss bridges are featured by replacing the conventional, separate cross-girder floor system and by a deck structure interacting with the main girders. This interaction alters the spatial rigidity of the truss bridge, and the stress pattern in the deck structure being rather complex, it is to be checked by measurements.

— Results of vibration measurements and of measurements of the dynamic effects in railway bridges pointed out several problems, so that, beside vibration measurements at some characteristic points of the main girder, also those of the deck structure and the wind bracings became imperative.

Experimental tests are intended to compare computational results and real behaviour, involving the analysis of static and dynamic load effects.

Loading was exerted by two coupled pairs of steam engine locomotives of the type 424 called a loading train (Fig. 3).

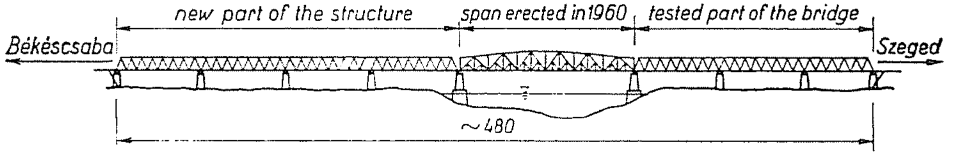

Fig. 1. General view of the bridge

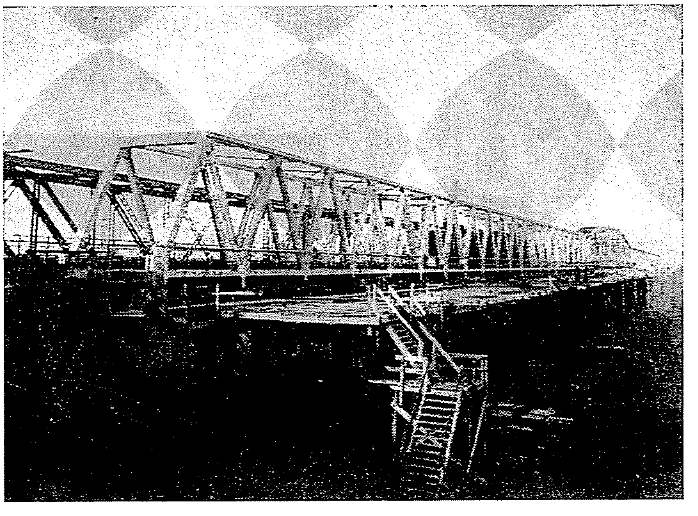

Fig. 2. The bridge across the Tisza river at Algyő

Fig. 3. Loading train on the bridge

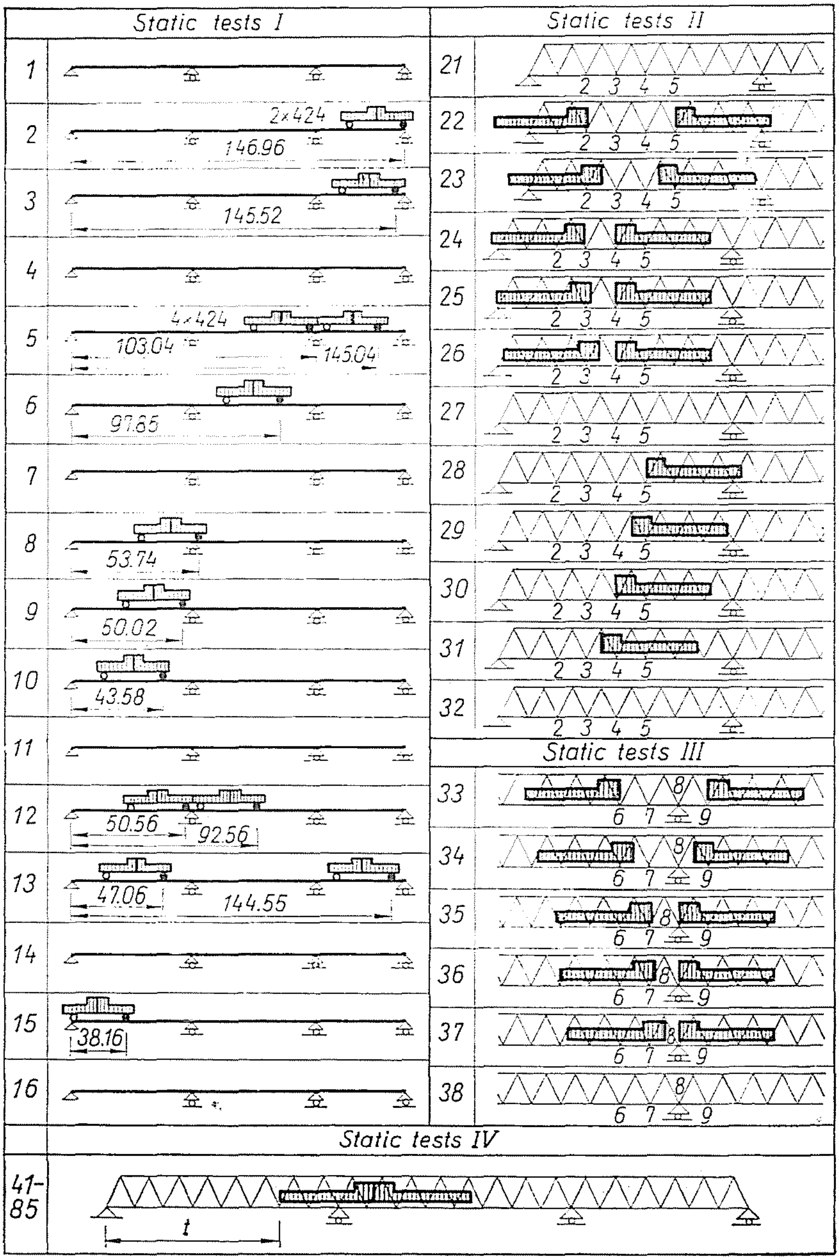

Fig. 4. **Test project**

2.1. *Testing program*

2.11. The static tests were carried out November 25 to 29, 1976. The program consisted of four parts (Fig. 4):

The *first* test series had to determine the characteristic deflections of the bridge upon the first loading.

Tests in the *second* series were meant to separate the effects due to main girder action from local action (See 3.1).

The *third* test series had to separate the quoted effect in the region of the support between spans VI and VII.

The *fourth* loading series aimed at determining the so-called "train influence lines".

2.12. November 29, 30 and December 1st, 1976, tests were made on the effect of moving load, again according to a program of four parts.

The *first* part aimed at determining the horizontal, transversal natural frequency of the bridge.

The *second* part determined vertical and horizontal displacements of the bridge.

The *third* part consisted in determining stresses at some typical points due to loading trains passing through the bridge at various speeds and in both directions.

The *fourth* part was an analysis of the effect of the normal traffic.

2.2. *Measurement methods and instruments*

Tests were planned to apply essentially the same sensors in both the static and the dynamic test programs for the measurement of strains and displacements of the structural members. Strains were determined by means of uni-, bi- and triaxial KYOWA strain gauges (made in Japan), displacements by inductive transducers of various types and measurement ranges (made by Hottinger Baldwin in F.R.G.).

Within the static program, the about 250 sensors were connected to 50-channel switching units on the bridge (Fig. 5), connected, in turn, to the measurement centre located in the bridge watchhouse comprising Hottinger Baldwin instruments completed by a Hungarian made digital voltmeter type MIKI, with serialiser, tape punch and typewriter.

Instrumentation for dynamic measurements is seen in Fig. 6. All selected transducers are directly connected to an amplifier, the output signals being continuously recorded by separate channels of the FM tape recorder.

As certified by built-in measurement checking points and field calibration, static measurement errors were within 20 μ/m, a rather favourable value, with respect to the difficult conditions (winter, all-day tests). Dynamic measurement errors ranged about 3%.

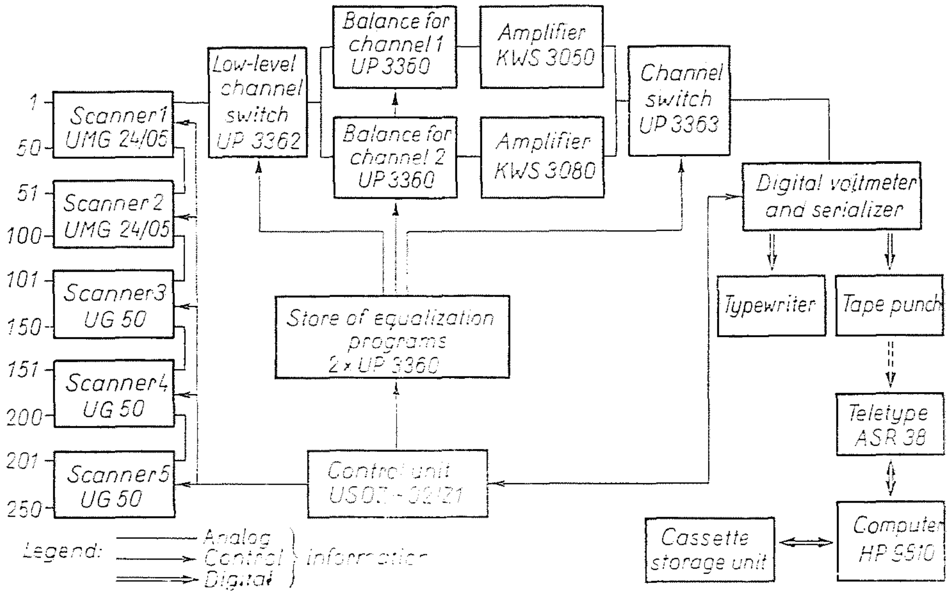

Fig. 5. Static test arrangement

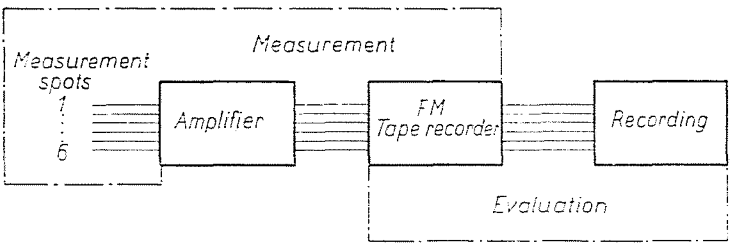

Fig. 6. Dynamic test arrangement

3. Static test results

3.1. *Normal stresses in the orthotropic deck plate*

In the actual case, the complex of bridge deck and supporting structural members (rail-bearer and cross girder, stiffeners in both longitudinal and transversal directions, box section below both deck edges) may be regarded as an orthotropic deck plate, essentially expected to support loading axles and to transfer the loads to the main girders. As a consequence, the orthotropic deck plate can partly be considered as a continuous beam elastically supported by cross girders, able to take direct local loads, and partly, as the bottom chord of the truss. Accordingly, its stresses may be attributed partly to the so-called local, and partly to the so-called main-girder action.

In the analysis, these two kinds of actions, to be treated as combined for load positions critical for stress maxima, are considered separately, applying reduced cross-sectional dimensions (effective width). Experiments are intended to confirm or correct the design assumptions and approximations.

Longitudinal normal stresses in the deck plate (parallel to the bridge axis) in four typical cross sections of the bridge are demonstrated in Figs 7, 8, 9 and 10.

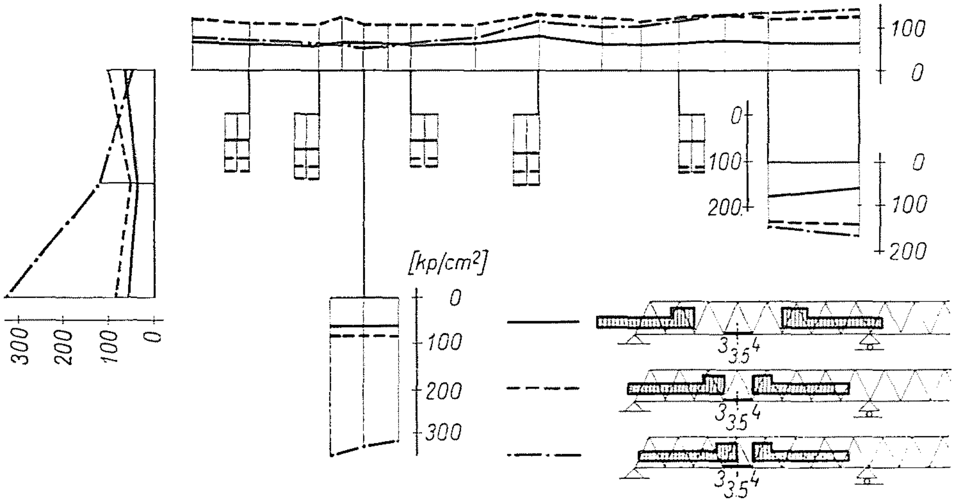

Fig. 7. Stress pattern at cross section 3.5

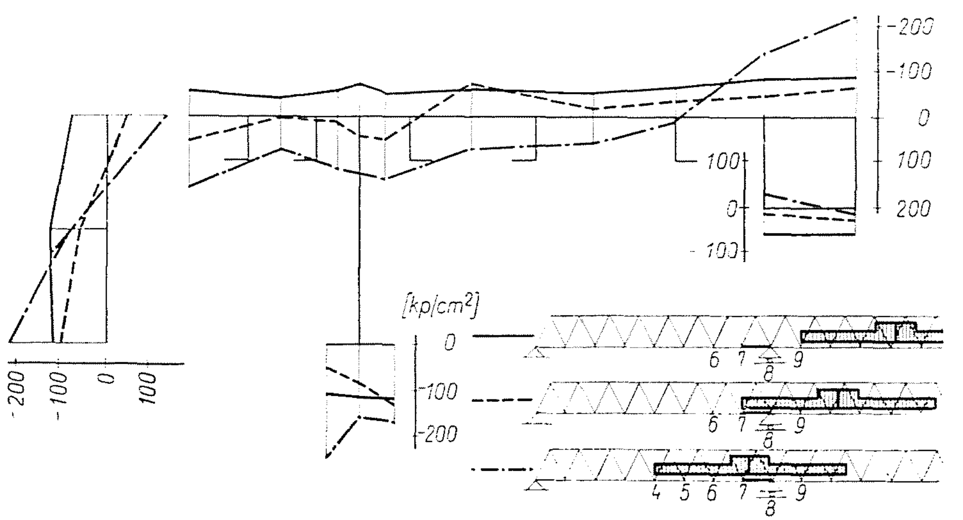

Fig. 8. Stress pattern at cross section 7

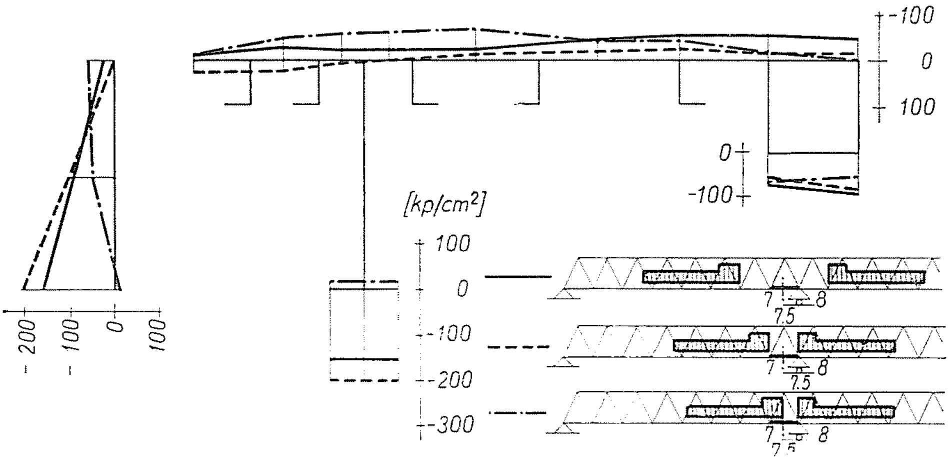

Fig. 9. Stress pattern at cross section 7.5

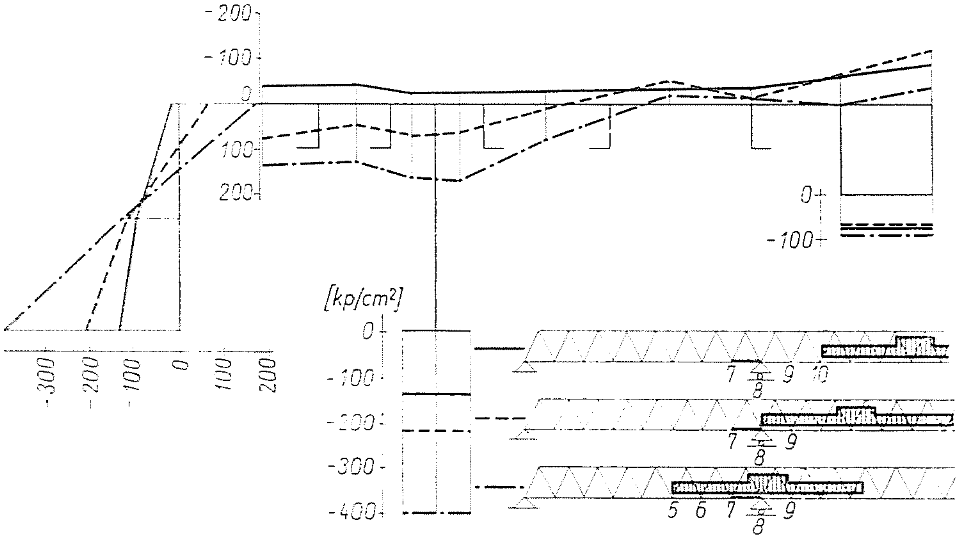

Fig. 10. Stress pattern at cross section 8

Actual deck plate behaviour will be illustrated by stress pattern in cross section 3.5.

Stresses traced in continuous line correspond to a load position where obviously local loading has little effect, hence the so-called main-girder action prevails. In this case, stress distribution may be said to be perfectly uniform. Thus, the entire width of the orthotropic deck plate as defined above may be considered as bottom chord of the main trusses, that is, in case of the main-girder action, the effective width is about the same as the total width.

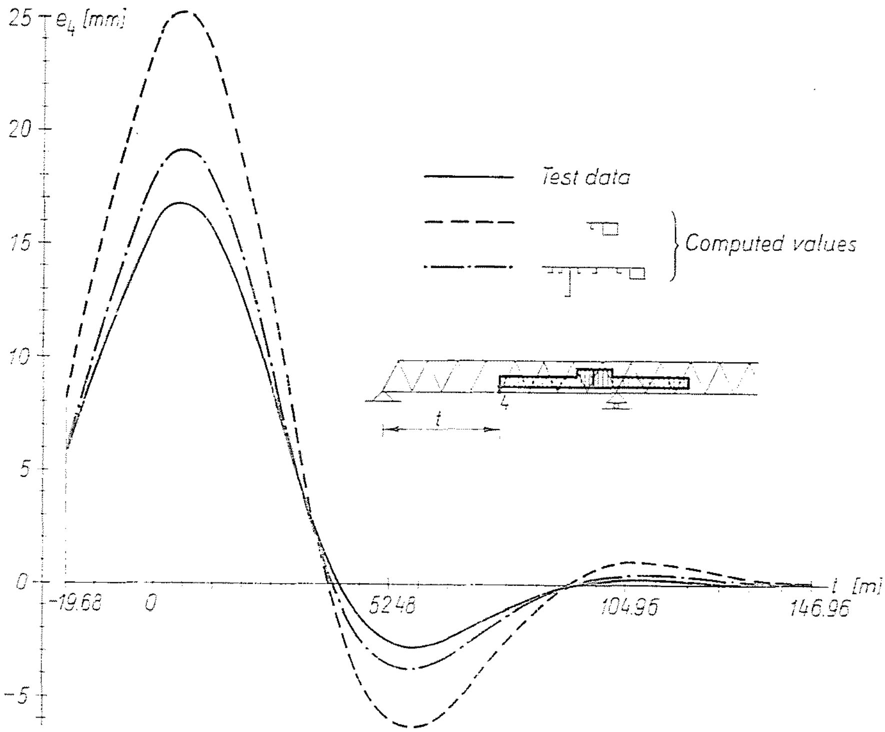

Fig. 11. Measured and computed deflections

Deflections of an outer mid-span are seen in Fig. 11. Actual deflections due to a loading axle group continuously moving along the bridge axis have been plotted in continuous line ("train influence line"). Dash-and-dot line indicates values computed with the total width. The relatively slight deviation is attributed to the somewhat higher rigidity of the truss than calculated because of the fixed-end diagonals. Such a deviation may be tolerated even in conventional trusses.

Considering the bottom chord of the main girder to be the edge strip of the orthotropic deck plate indicated in the figure (effective width being about 40% of the total width), deflections traced in dashed line result. There is a striking deviation from the real values. For hyperstatic trusses, cross-sectional area of the diagonals affects the magnitude of the deflections and forces as well. Although forces calculated with cross-sectional areas assuming different effective widths generally deviate by less than do deflections, they are not negligible in case of certain members (bottom chord). Let us notice that even the actual forces in the members determined from measured stresses were closer to forces calculated according to the dash-and-dot line.

The orthotropic deck plate acting as the top flange of a continuous beam for taking and transmitting loads is characterized by stresses traced in dash-and-dot line in Fig. 7. Subtracting the computed tensile stresses of the bottom chord (due to the main-girder action) from the extreme fibre stresses at the rail bearers results in

$$\sigma_f = -80 \text{ kp/cm}^2, \qquad \sigma_a = +200 \text{ kp/cm}^2$$

in the upper and lower fibre, respectively. Considering them the extreme fibre stresses of a beam, the effective width of the deck plate corresponding to the local action can be determined, in the actual case

$$B = 900 \text{ mm}.$$

Stress distributions in cross sections (7; 7.5; 8) adjacent to the support essentially confirm the above, with the following differences:

— Extreme parts of cross sections (7; 8) beside the gusset plates exhibit nodal secondary stresses due to bending moments in the truss plane.
— Stress values in the bottom flange of the rail-bearer hint to eccentricities.
— Member 7—8 is a compressed bar of the truss, therefore the direct bending due to the out-of-straigthness of the deck has a sensible effect that could, however, be determined in two points of the cross sections only, using strain gauges mounted bilaterally of the deck plate.

Stresses due to a given load position in different cross sections of section 7—8 are illustrated in Fig. 12.

3.2. *Normal stresses perpendicular to the longitudinal bridge axis*

Normal stresses in the bottom and top flange of the cross girder joining node 7 are seen in Fig. 13. Taking also other, related measurements into consideration, our tests permit to consider the cross girder as a simple beam with partially fixed ends. Remind, however, that the rather intricate interaction between structural parts of the orthotropic deck plate system makes their description as separate beams rather difficult, especially for the cross-girder.

3.3. *Stresses in the diagonals*

Tests were made on a box-section and on a I-section tensile diagonal. Axial normal stresses measured in an intermediate cross section of diagonal 7-VII, along its clamping, and in the gusset plates are seen in Fig. 14. Stress distributions in cross sections lead to the following conclusions:

— Nodal secondary stresses due to the bending moment in the truss plane and normally to it are clearly seen in an intermediate cross section of the diagonal and in the clamping cross section.

— Stresses in cross sections near the clamping are much higher in plate components (flanges) of the diagonal parallel to the gusset plates, hence, in the force transfer plane they are much higher than in the webs. This effect, as before, adds to the nodal secondary stresses.

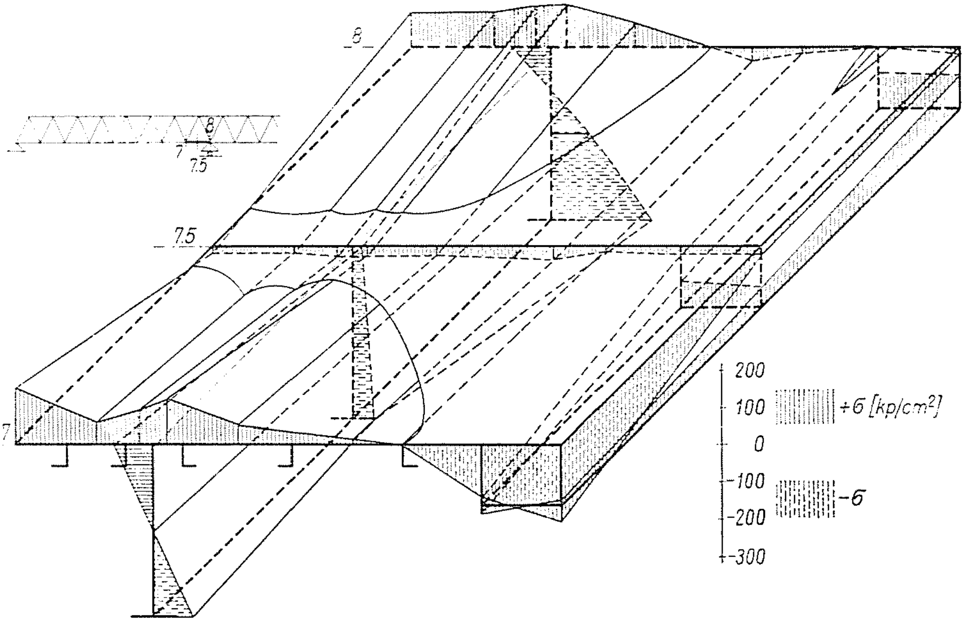

Fig. 12. Stress pattern in the bridge deck

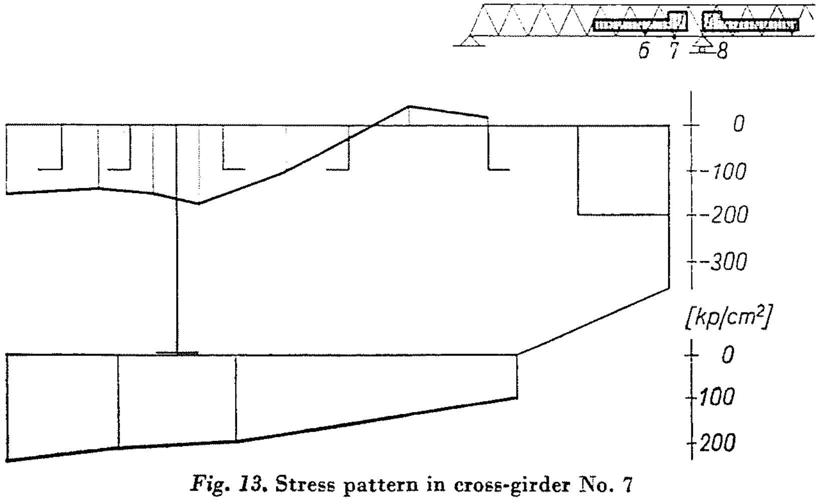

Fig. 13. Stress pattern in cross-girder No. 7

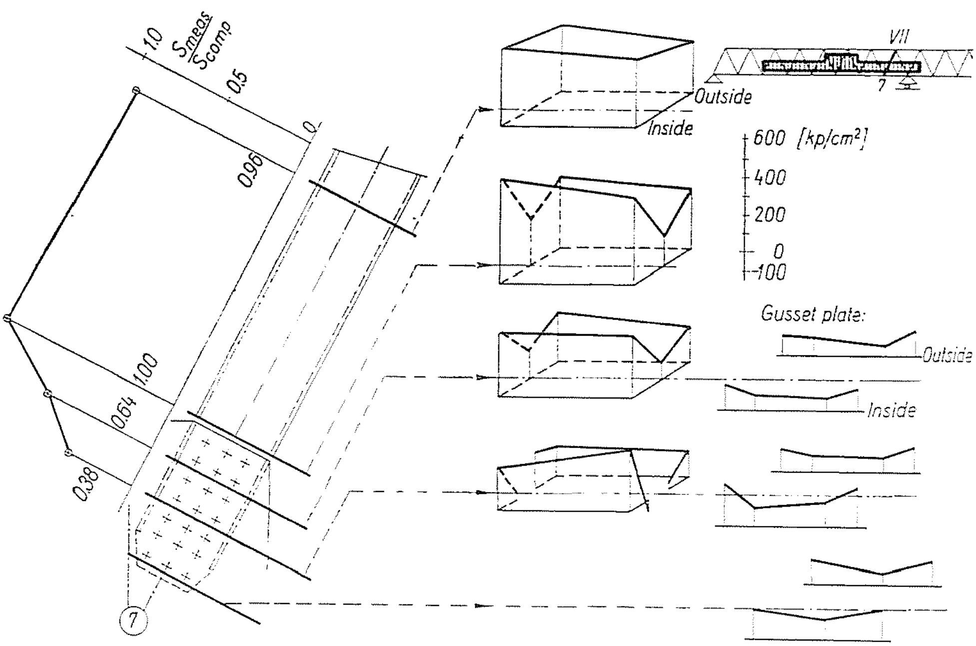

Fig. 14. Stress pattern in diagonal S_{7-VII}

— Axial variation of the bar force calculated from measured stresses have been traced in a continuous heavy line. Although the first rows of the relatively long prestressed bolted connection (of seven rows of bolts) are seen to transfer more force, forces taken by bolt rows do not greatly differ.

— Gusset plates exhibit no stress concentration maxima or other disorders.

The above are invariably true for I-diagonal 3-II. Stress pattern in diagonal cross sections and bar force variations are seen in Fig. 15.

The knowledge — at least approximately — of nodal secondary stresses is of utmost importance for the design of railway truss bridges. From this aspect, actually, test methods determining the actual behaviour of the structure are of paramount importance. Figs 16 and 17 represent cross sections of the two tested diagonals adjacent to the clamping. Mean stresses in the cross sections have been plotted in a continuous line, secondary stresses due to bending moments in, and normal to, the truss plane have been traced in dashed, and in dash-and-dot lines, respectively, in form of the quoted "train influence line". The stress maximum in the cross section is the sum of the three curve values at the same abscissa. As for the secondary stresses, these do not exceed 150 kp/cm² even in the worst case. Notice that the test load about equalled the service load for fatigue design specified in the draft of the Hungarian Railway Bridge Codes 1976.

3.4. *Displacements in prestressed bolted connections*

Rather few data have been published on the displacements in prestressed bolted connections in existing structures. Therefore relative displacements of flange and gusset plate of the two quoted diagonals 7-VII and 3-II were measured and plotted in Figs 18 and 19, respectively, again as "train influence lines", except that here values for unloaded condition, in fact, slight slips, have been inserted between the data of loading series. Because of the rather slight displace-

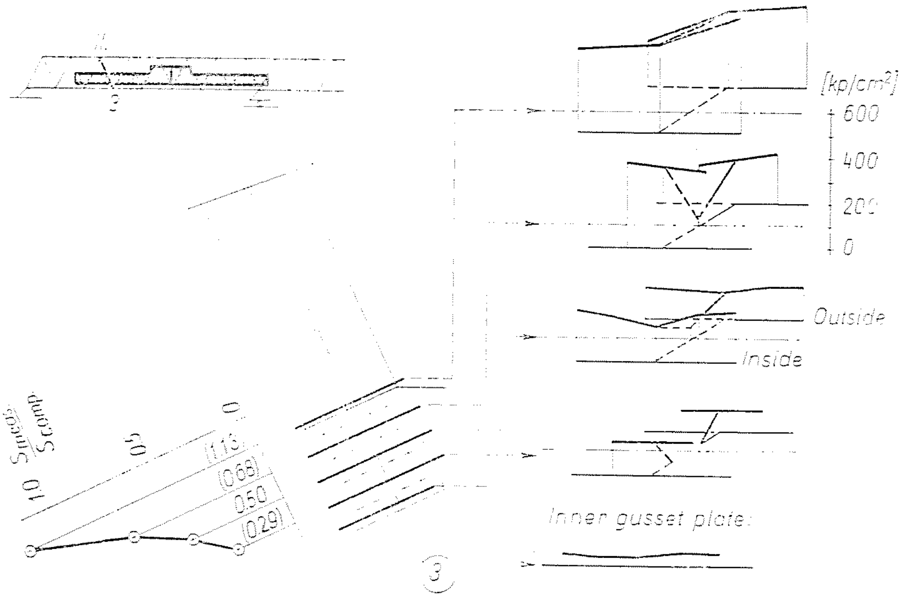

Fig 15. Stress pattern in diagonal S_{3-II}

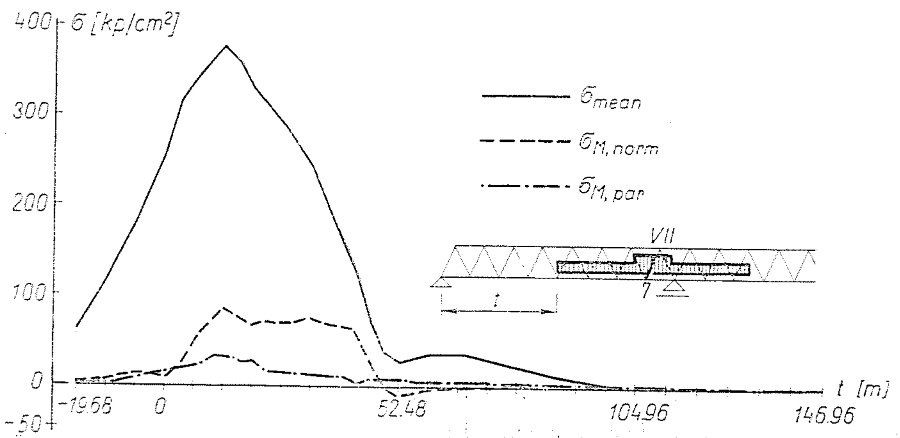

Fig. 16. Secondary stresses at the connection of diagonal S_{7-VII}

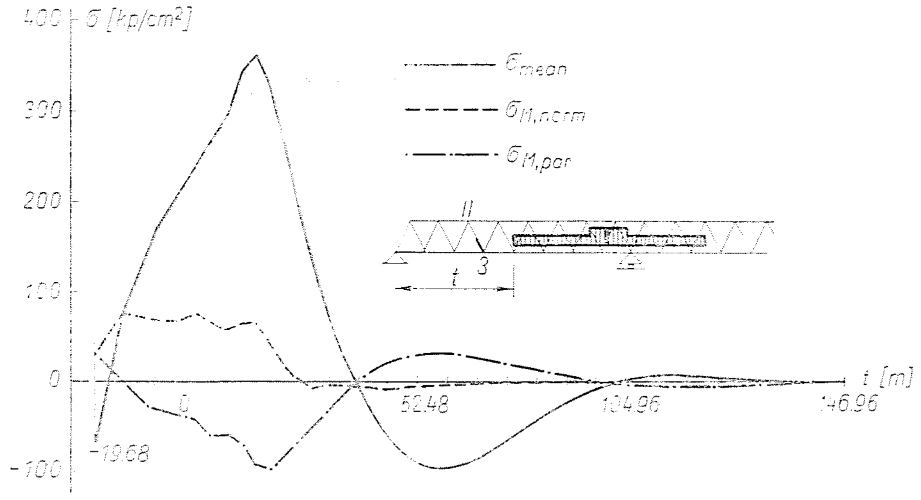

Fig. 17. Secondary stresses at the connection of diagonal S_{3-II}

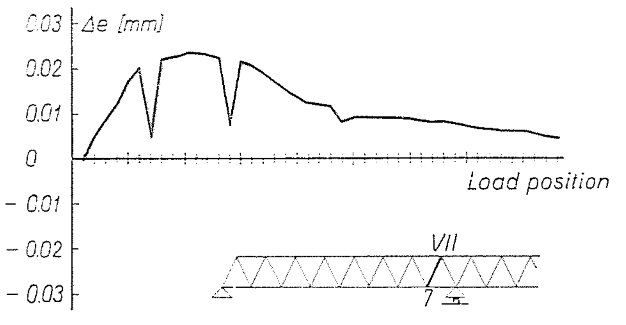

Fig. 18. Elastic displacement at the connection of diagonal 7-VII

ment values and the prolonged measurements, accuracy of measurements is much poorer than for other displacement measurements, but comparison with Figs 16 and 17 shows joint displacement curves to be proportional to bar force diagrams. Remind also that the plotted displacements also contain elastic displacements of diagonal sections; these values nevertheless are lower by an order of magnitude than the plotted ones.

3.5. *Other tests*

In addition to those described above, the following static tests have been made:

— bridge bearing impaction measurements,
— examination of longitudinal displacements of the bridge structure,
— examination of bridge end bearing rotations,

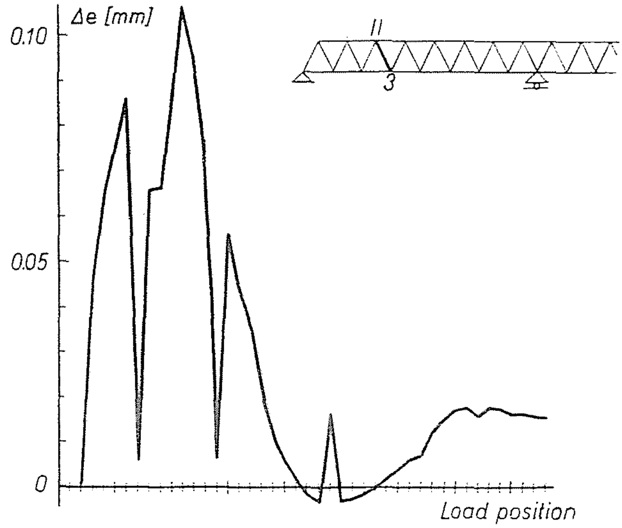

Fig. 19. Elastic displacement at the connection of diagonal 3-II

— stress distribution in the deck plate adjacent to the gusset plate,
— shear stresses along the connection of the rail-bearer.

Because of space shortage, these latter cannot be detailed here but see in [1].

4. Investigation of the dynamic effects

Tests aimed at determining dynamic effects due to rolling loads in the entire bridge structure and in its main structural parts; at confronting measured and design values according to specifications. Also possibility of "lasting" resonance phenomena upon loads rolling at different speeds, likely to be injurious to the structure or its parts, has been investigated.

In designing the bridge, additional stresses due to moving loads have been assumed with a dynamical factor specified in the 1951 Railway Bridge Codes, giving an approximate formula for the relationship of dynamic factor and span — similar to other national codes.

After 1951, the UIC (International Railway Union) sponsored important international research work to determine dynamic effect as exactly as possible. In its final report, UIC gave recommendations for the dynamic factor [2]. Accordingly, the dynamic effect depends on the span length, the vehicle speed and the natural frequency of the bridge. Our tests involved comparison of the measured stresses to the recommended ones. This was not possible for the tentative draft of the Hungarian Railway Bridge Code in force since August 1st, 1976, where — as against UIC recommendations requiring high-accuracy dynamic factor determinations — the effect of moving loads has been involved in a tabulated "train factor", including different additional vertical load actions, preventing the dynamic effect from being separated.

Dynamic tests involved some 40 measurement locations selected from among displacement transducers and strain gauges used in static tests so as to be typical of the dynamic behaviour of the major structural members. The dynamic factor had been determined as quotient of the maximum dynamic excess by the maximum static stress. Our measurement results for the main girder (diagonal), the cross girder and the rail-bearer are seen in Figs 20, 21 and 22. Measured dynamic factor values are invariably seen to closely approximate the average UIC recommendations and to be within the double standard deviation range of this average. Also the dynamic factors specified in the 1951 Railway Bridge Code are displayed.

In cases of the main girder, the rail-bearer and the cross girder, determination of the dynamic factor permits to describe stress excesses due to moving loads. Nevertheless on the deck plate — especially when measuring transversal stresses — stress excesses are higher than the — usually rather low — static maxima (20 to 50 kp/cm^2). Thus, it seems here advisable to indicate the absolute value of the dynamic stresses. The character of stresses is illustrated in Fig. 23 presenting the stress development measured by two transversal and one longitudinal strain gauges on the deck plate, and by one longitudinal

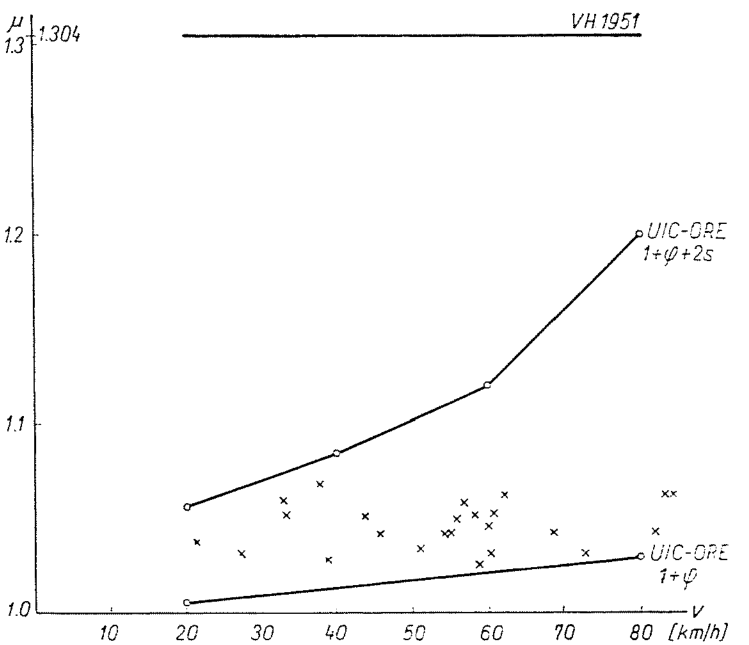

Fig. 20. Dynamic effect on the main girder

strain gauge on the bottom flange of the rail-bearer. By the way, the quoted measurement showed the highest stress excess in the deck plate: a stress wave spanning 320 kp/cm² peak to peak at measuring location No. 206. Vibrations of a relatively high frequency (50 to 80 cps) are observed in the figure to be mostly transversal ones, and to be of short duration in longitudinal measurements of rather steady development.

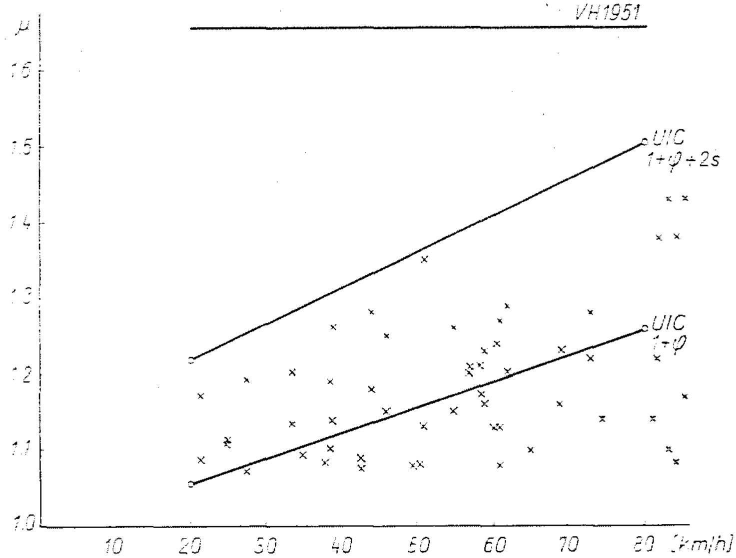

Fig. 21. Dynamic effect on the cross girders

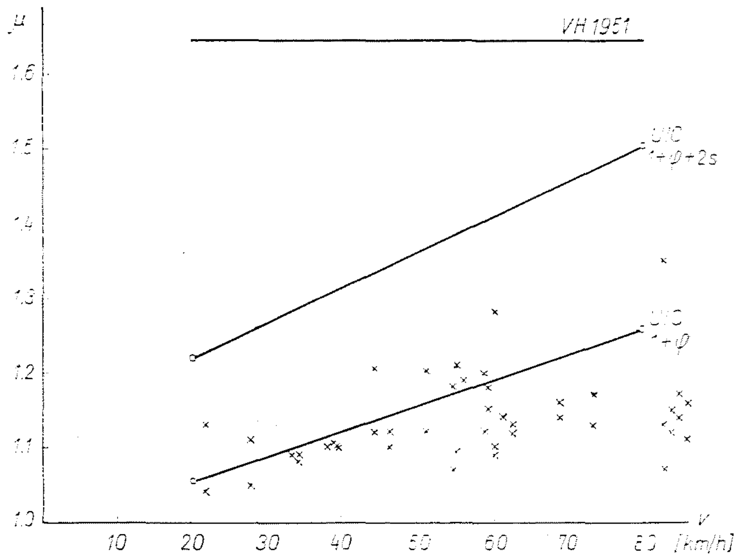

Fig. 22. Dynamic effect on the floor girders

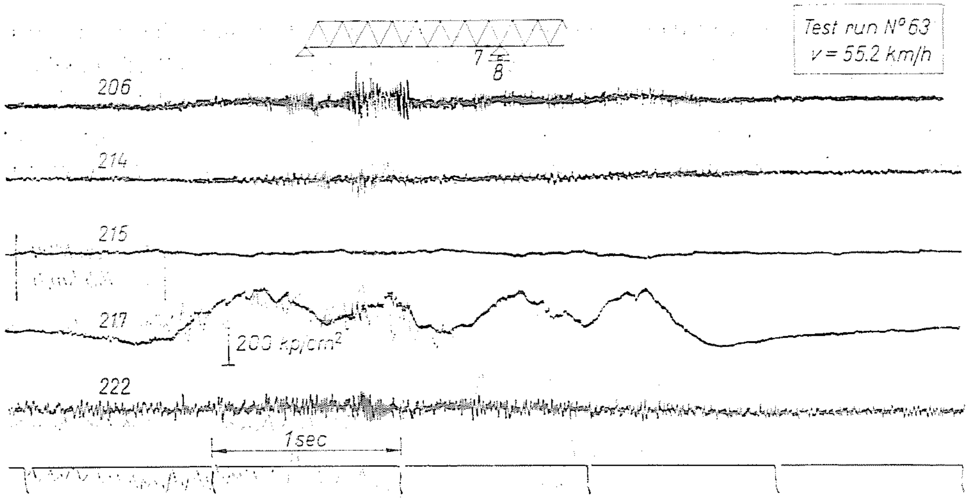

Fig. 23. Dynamic effect on the bridge deck
Allocation: transducer No. 217 on floor girder (lower flange), others on the deck plate

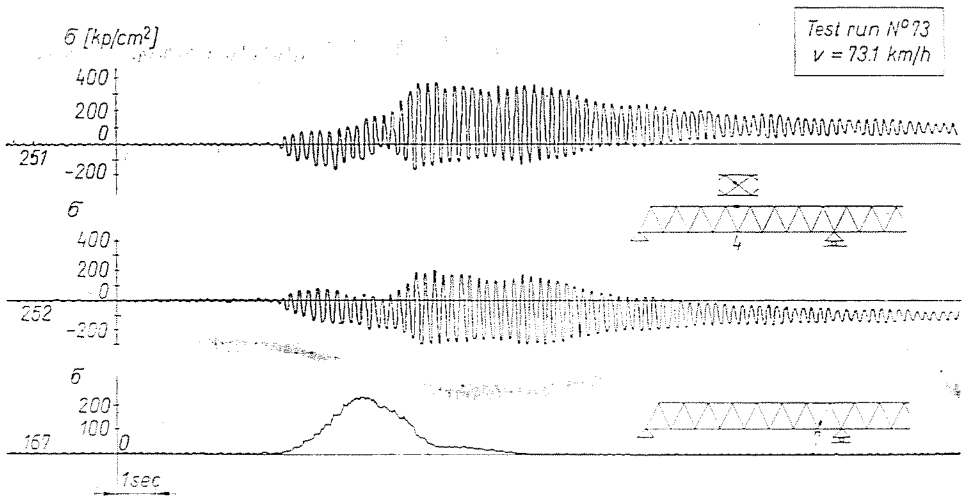

Fig. 24. Vibration in wind brace and diagonal

Like for deck plate stresses, also for the wind brace stresses the dynamic effect dominates. Figure 24 shows high dynamic effects to be superimposed to rather low static mean stresses. Wind brace stresses have been recorded for about 30 train passages at different speeds. In high-speed passages (70 to 80 km/h), the peaks were as high as ± 300 kp/cm², vibration frequencies ranged from 3.91 to 4.35 cps, independent of the passage speed. This fact, and the vibration pattern itself, hint to a resonance phenomenon, with a maximum effect where the excitation frequency coincides with the wind brace natural frequency. The excitation was attributed to the "hammer blow" of wheels of passing steam locomotives. In this case, the speed range of 70 to 80 km/h corresponded to a frequency of 3.9 to 4.3 cps. In fact, this was the range where dynamic effect maxima have been recorded in the wind brace. If this assump-

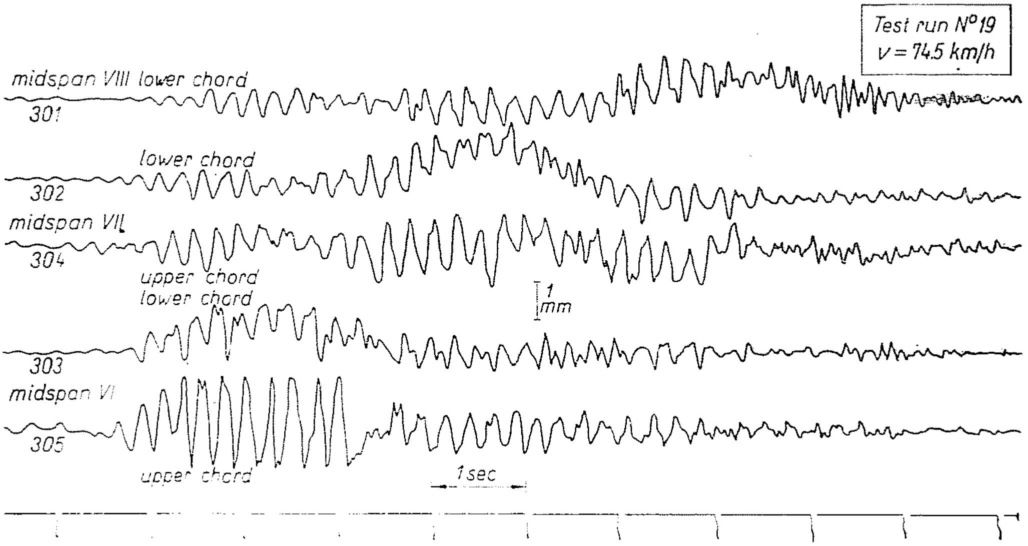

Fig. 25. Horizontal mid-span vibrations

tion were correct, speeds over 90 km/h would again result in decreasing vibration amplitudes. Unfortunately, the condition of the joining track prevented us from performing relevant measurements.

Similar resonance phenomena were observed in horizontal, lateral displacements measured on the structure. Displacement transducers were mounted at each mid-span, on bottom and top chords. Displacements had absolute values invariably below 1.5 mm, much lower than the 1/5000 part of the span specified in the 1976 Hungarian Railway Bridge Code. It was interesting, however, to observe resonances — even if instantaneous — in bridge structure sidesway. On Fig. 25, e.g. the constant-amplitude vibration of the top chord of span VI had a frequency of 4.17 cps at an exciting frequency of 4.13 cps. Wind brace vibrations and bridge sidesway had identical or little different frequencies. All our measurements pointed to the non-parallelity between bottom and top chord motions, hinting to the distorsion of the cross section during load passage. All these point to the importance of studying the problem of sidesway, neglected up to now.

Summary

The Department of Steel Structures, TU, Budapest carried out investigations with a newly built, welded single track continuous railway truss bridge, with orthotropic deck plate and high-tensile prestressed (HT) bolt connections. Investigations involved both static and dynamic tests to compare computational results to real structural behaviour. Description of tests and the most important experimental data are given, together with their comparison to results of analyses according to Hungarian and international design codes. Experimental and theoretical investigations contributed to clearing up problems as the choice of computational model for stresses in the orthotropic deck plate and main girders; force-pattern and displacement in HT bolt connections, secondary stresses in diagonals; natural frequencies, dynamic effects and resonance phenomena in the main parts of the bridge.

References

1. Research Report on the Experimental Analysis of the Three-Span Structure of the Railway Bridge across the Tisza River at Algyő.* Department of Steel Structures, Technical University, Budapest, 1977.
2. Determination of Dynamic Effects in Bridges. UIC-ORE Final Report, Utrecht 1970.

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