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ANALYSIS OF THE FALL-DOWN OF THE BRIDGE OVER THE HERNÁD AT GESZTELY

P. PLATTHY

Department of Steel Structures
Technical University, H-1521 Budapest

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Abstract

In Hungary, several old trussed highway bridges of trough-type fell down because the vehicle passing through them was higher than permitted and got caught into the upper wind-bracing. The analysis shows that the trussed bridges where upper wind-bracing can be found only in the middle section of the bridge (the height of the trussed girder is varied) are liable to be destroyed by vehicles of a high load.

Introduction

In Hungary, for economic reasons there are a number of highway steel bridges which do not comply with all the traffic demands of the highway going through it, with respect to clearance gauge or the load-carrying capacity. In these cases, road signs located in front of the bridges draw the attention to the prescription that vehicles of what height or weight are permitted to run through them. However, it has been proved that the prohibition by road signs has generally not reached the required effect.

In the last 20 years, several highway bridges were seriously damaged in Hungary, and four highway bridges collapsed completely because the vehicle was higher than permitted to pass through. The case of the bridge over the Hernád at village Gesztely is characteristic of the cases of collapse. There was an upper wind-bracing mounted on the middle section of the trough-type, trussed steel bridge of 46 m span built several decades ago. In October 1988, a lorry hauling an excavator was passing through the bridge. The jib of the excavator got caught in the first transverse wind-bracing, and the bridge fell down by the effect of impact (*Figs. 1* and *2*). The estimated damage was 60 million forints. To clear up the causes of the catastrophe, an inquiry was ordered officially whose certain details or results, respectively, will be described in the following.

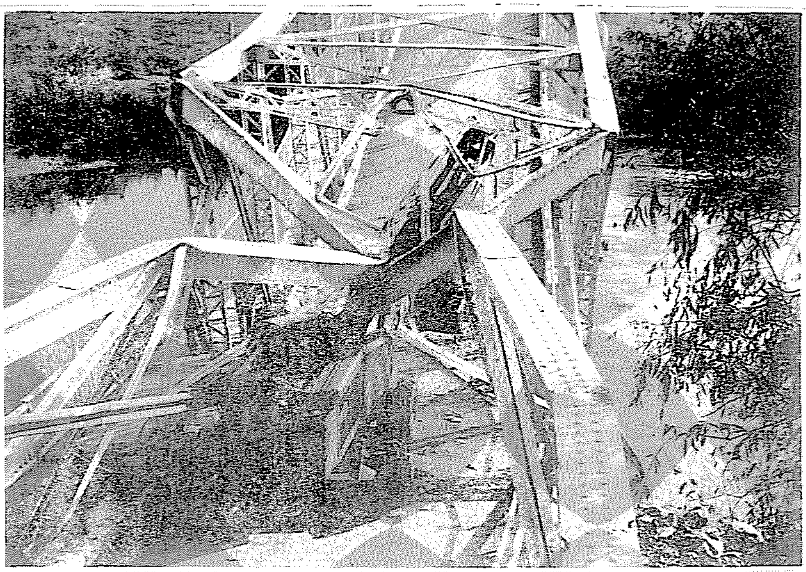

Fig. 1.

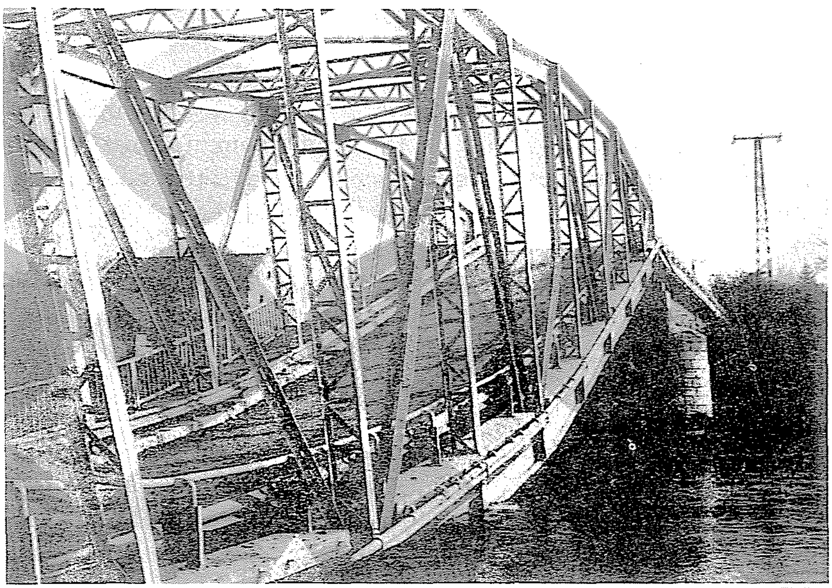

Fig. 2.

On-Site Inquiry and its Results

Statements Made at the On-Site Inquiry

In the course of the on-site inquiry held on the wrecks of the bridge over the Hernád at Gesztely, the following statements could be made:

a) The collapsed bridge was an old type trough structure, designed with the use of principles not applied any more in today's practice, and constructed with riveted trussed main girders (*Figs. 1* and *2*).
b) The accurate lifetime of the main girders of the bridge could not be determined by visual examination. It is only sure that the main girders contained also such rolled profiles which were not manufactured after the World War II any more.
c) The floor structure of the bridge was rebuilt in 1984. The floor structure (floor beams, the reinforced concrete floor slab, etc.) was in a faultless condition at the time of collapse and — according to the on-site inquiry — its condition could not cause the catastrophe. The damage to the welded longitudinal girders and the reinforced concrete slabs occurred at the time when the load-carrying capacity of the main girder ceased, and the bridge collapsed.
d) The main girders of the bridge were subject to repair and repainting in the course of rebuilding of the floor structure.
e) the time of the on-site inquiry, only the excavator was left behind on the bridge, by the lorry was hauled away in the meantime. The excavator could not be removed without demolishing the structure, because the structural material got entangled in the jib of it. It could be stated unambiguously that the jib got caught in the first transverse bracing of the upper wind brace and, as a result, the upper chord was contracted by the tensile force arisen in transverse bracing (*Fig. 1*).
f) The expansion shoe of the collapsed bridge slid off the abutment, and the bottom chord of the structure was lowered to the bottom of the river bed.
g) The impact against the river bed bottom caused further damage to the structure. The bottom chord became crooked due to leaning on the river bed bottom, and so the upper chord, too, suffered deformation. Due to the distortion of the chords, some truss members underwent buckling.
h) The excavator left behind on the bridge wrecks was damaged,. and the individual elements of it underwent well-observable, permanent deformation.

Reproduction of Failure Process

On the basis of the experience gained from the on-site inquiry, the failure of the bridge can be summarized as follows:

a) The lorry hauling an excavator entering the bridge from the direction of Miskolc (Onga) overpassed the cross-sectional area at the fourth post of the trussed main girder where the transverse bracing of the upper wind brace was laid out (*Fig. 3a*) while the jib of the trailed excavator got caught in this transverse bracing. By the effect of impact, the transverse bracing deflected the upper chords of the main girder together with the posts assembled from their original plane (contraction), as it can be seen in *Fig. 1*.

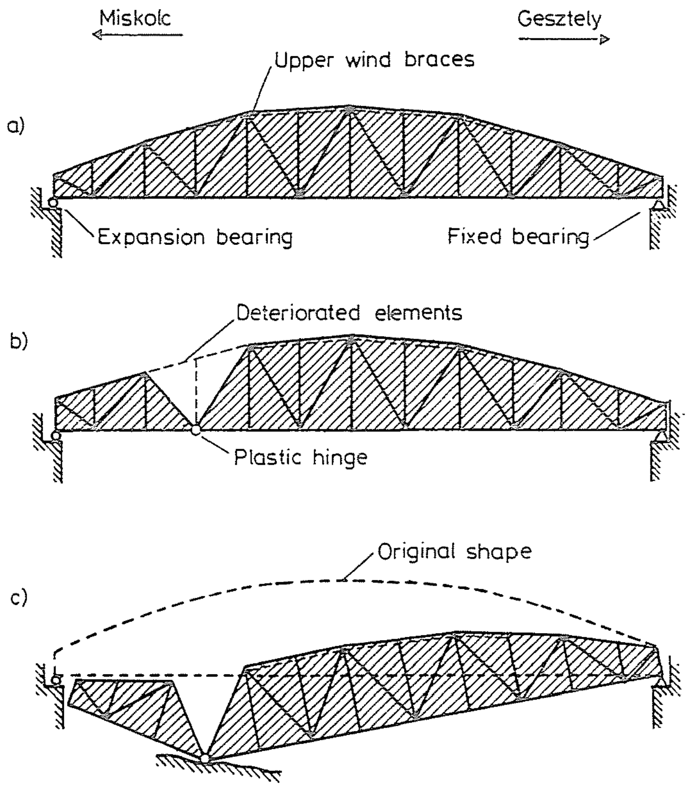

Fig. 3.

b) The chords of the main girder deflected from their designed centre line — since a compressive force worked in them necessarily by the effect of deadweight and the weight of the hauled vehicle — lost their load- carrying capacity (they underwent plastic buckling) and so did the posts, too. Consequently, the elements (bars) traced in dashed line in *Fig. 3b* did not work any more. This involved the consequence that, in this place, both the joint of the bottom chord, and the floor structure were overloaded, which — in turn — led to the development of a plastic hinge (as indicated in *Fig. 3b*).

c) The structure becoming a kinematic chain (unstable bar chain) due to the plastic hinge collapsed slowly under the influence of the loading weights (deadweight, lorry, excavator), the girder end at the bearing-support came off the abutment, and the section of the bottom chord to which a change in direction was brought about due to the plastic hinge was leaning on the bottom of the river bed (*Fig. 3c*). In this position, the structure — as pieces of 2 two-support beams — was held in equilibrium.

Exclusion of Other Causes of Failure

Other Possible Causes of Failure

If the experience gained from the on-site inquiry is disregarded, i. e. the fact that the collapsed bridge structure was in a well condition, and that the first transverse bracing of the upper wind-brace got entangled in the jib of the excavator — which undoubtedly proved the catch-in of the jib —, then as other causes of collapse, the following could have been mentioned:

- use of non-adequate material, or
- overload or fatigue, respectively.

Thus, examination of the causes mentioned above seemed reasonable.

Testing of Materials

Since the failure occurred at the main girders of the bridge, only their material was to be checked.

According to the tests performed on the samples of the material cut out of the bridge wrecks, the used profiles were made of a high-grade open-hearth steel (Martin steel) of grade 37.

The results of the tensile tests and Charpy tests (impact tests) fully satisfied the requirements stipulated in the Hungarian standards. Consequently, the cause of the failure could not be the insertion of a weak material liable to breakage.

Possibility of Overloading or Fatigue, Respectively

The possibility of overloading was subject to examination by applying force-theory calculations. The examination — since the road sign placed in front of the bridge indicated that the structure was classified as 'C' group with respect to dimensioning — covered, in addition to the load acting at the moment of collapse, also the case of load 'C' stipulated in the Hungarian standards.

The deadweight of the structure — on the basis of the detailed analysis — as related to one main girder was:

$$g = 23.65\,\mathrm{kN/m}\,,$$

while the data of the vehicle passing through at the time of collapse, as well as the network of the main girder are shown in *Fig. 4*.

The dynamic factor used in calculations according to the Hungarian specifications was:

$$\mu = 1.15$$

in each case.

According to force-theory calculations, the maximum calculated compressive stress arisen in the upper chord No 3–5 of main importance with respect to the failure of the structure — if load 'C' is taken into consideration — was yielded as:

$$\sigma_c = 6.39\,\mathrm{kN/cm^2}\,,$$

while in case the real vehicle was taken into consideration, it was yielded as:

$$\sigma_j = 4.70\,\mathrm{kN/cm^2}\,.$$

These stresses are essentially smaller than it could have been permitted for the given structure on the basis of the slenderness of the bar, or the rigidity of frame, respectively. Thus, e. g. the permissible stress for a bar of symbol 5-d on the basis of slenderness would have been:

$$\sigma_{\mathrm{perm}} = 13.39\,\mathrm{kN/cm^2}\,,$$

while with the bar of symbol 3-d, the required frame rigidity in the case of reckoning with the load causing the collapse was yielded as:

$$F_{\mathrm{requ.}} = \frac{4S}{a\nu^2} = \frac{4.941}{405.8 \cdot 3^2} = 1\,\mathrm{kN/cm}\,,$$

and the available frame rigidity was yielded as:

$$F_t = \frac{2.1 \cdot 10^4}{\dfrac{470^3}{3 \cdot 16605} + \dfrac{503^2 \cdot 640}{2 \cdot 90005}} = 10.08\,\mathrm{kN/cm}\,.$$

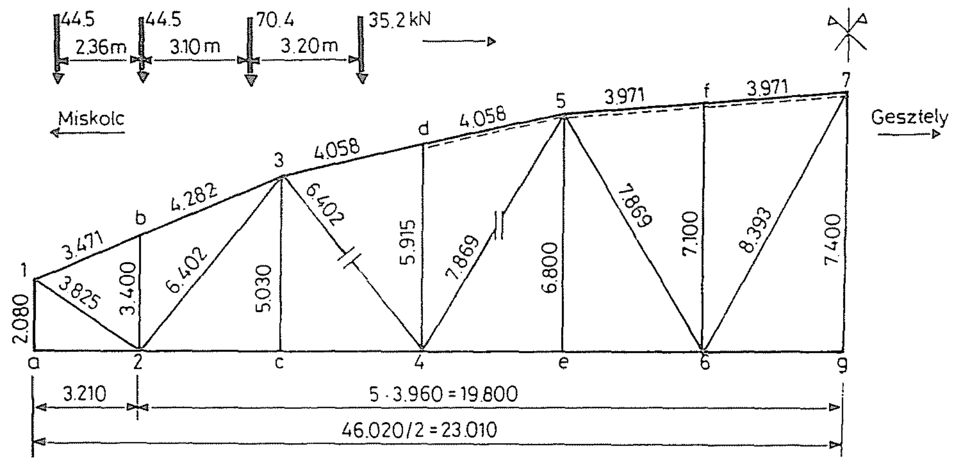

Fig. 4.

Consequently, the failure of the bridge could not have occurred due to the loss of stability in the case of a normal service.

The fatigue test of the structure seemed to be superfluous according to the stipulations of MSZ-07-3702. Since — according to the stipulations — the permissible stress for fatigue with a riveted structure will be:

in the case of tension: $\sigma_{f,\mathrm{perm}} = 8.0\ \mathrm{kN/cm^2}$,
in the case of compression: $\sigma_{f,\mathrm{perm}} = 13.3\ \mathrm{kN/cm^2}$.

However, at the same time, in the course of fatigue-test a smaller load should be reckoned with than in the course of stability test mentioned above. *Consequently, it can be stated that the structure was not underdimensioned with respect to fatigue* (neither the on-site inquiry detected the signs of fatigue).

Finally, it is worth pointing out the fact that according to the examination performed by means of calculation on the plastic hinge developed on the bottom chord, the moment effecting the development of the plastic hinge and the ultimate moment — if the resistance of the floor beams and the reinforced concrete floor slab are also taken into consideration —

were close to each other. In this way, the fracture occurred in the floor structure could be developed only relatively slowly consuming a little more time, which can be considered a favourable fact because the occurrence of an abrupt collapse could have endangered even the life of the driver. An abrupt collapse can be brought about only by a brittle fracture (rigid breakage) or a rapid loading greater at least by one order of magnitude than the ultimate moment.

Analysis of Failure

Kinematic Test of the Mechanism Developed During the Failure of the Structure

Figs 5a and *5b* show the top view of the structure in the state prior to failure, and its state during the failure process, respectively.

In *Fig. 5c,* the side elevation of the structure (main girder) can be seen in the state prior to failure, and during the failure process, respectively.

The geometrical data indicated in *Fig. 5* (t_0, t, l_1, l_2, k, v, u, w, z) are correlated with each other. If the change in length of the elements is disregarded (elongation, compression), then the relationships can be determined relatively in a simple way, and the motion of the structure during failure (kinematics of the developed mechanism) can be described or examined mathematically.

It is reasonable to examine the behaviour of the mechanism as a function of the vertical displacement (z) of the plastic hinge brought about on the bottom chord. The data calculated in this way are contained in *Table 1* and *Fig. 6,* respectively.

The results obtained by means of the simplified kinematic model are in harmony with the state experienced during the on-site inquiry. Thus, e. g. the end towards the bearing support of the bridge had to fall from the abutment.

Stability of the Chord in Compression in the Case of Transverse Force

As it could be stated also in the course of the on-site inquiry, the jib of the trailed excavator got caught in the upper transverse bracing and, as a result, the contraction of the upper chords was brought about by a force Q (*Fig. 1*). It is problematic what magnitude of tensile force Q_t could take an effect eliminating the resistance (load-carrying capacity) of the chord.

At the instance of catch-in, a compressive force $N = 941$ kN was acting on upper chord 3–5, according to our calculations. In case this value, as

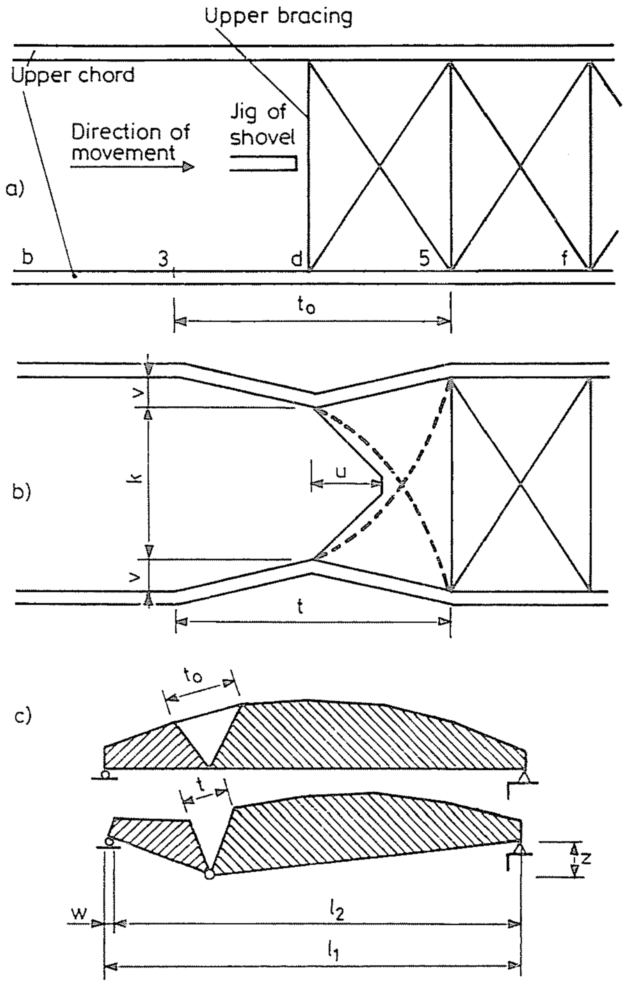

Fig. 5.

well as the dimensions of the structure are taken into consideration, and an ideally elastic-plastic material is assumed, as well as a theory of the second-order is used, then the relationship between contracting force 'Q' and transverse displacement 'e' (in *Fig. 5b,* the displacement indicated by 'v') can be determined for the two extreme cases, particularly with respect to:

- the chord of hinged support at both ends, or
- the hingeless chord, respectively.

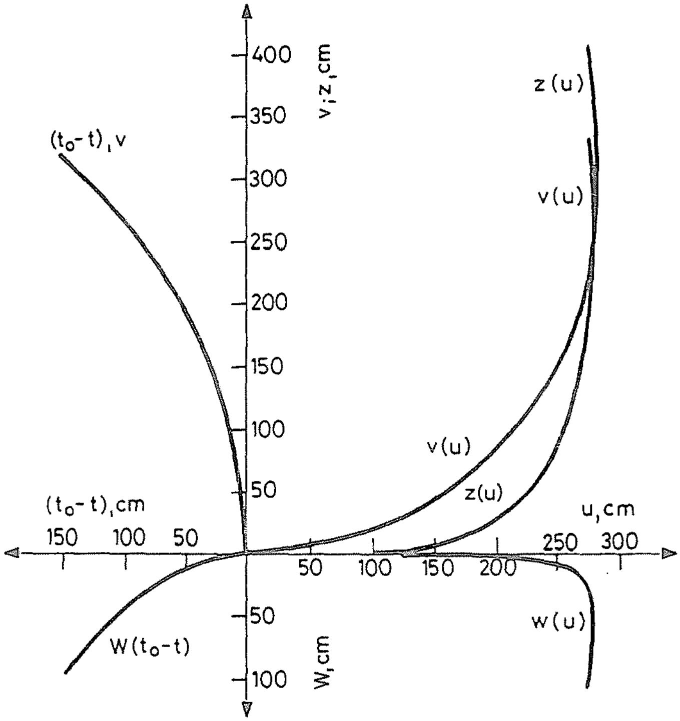

Fig. 6.

The results of the approximate calculations of this type performed by means of stability functions stipulated in MSZ 15024/4–85 are shown in *Fig. 7.* Since the reality is closer to the case of the clamped bar (the intermediate post, too, reinforces the chord to some extent, though it is not too considerable according to the numerical examinations), on the basis of *Fig. 7,* it can be accepted as an approximation that the stability of chord 3–5 occurred by the effect of transverse tensile force:

$$Q_t = 260\,\mathrm{kN}\,,$$

and a lateral displacement:

$$e = 6\,\mathrm{cm}\,.$$

With such deflection, the displacement of the upper transverse bracing designated by symbol 'u' in *Fig. 5b* (see also *Fig. 6* and *Table 1*) will be:

$$u = 58\,\mathrm{cm}\,.$$

Table 1
Displacement of the kinematic chain of the structure in cm dimensions

z	u	v	$t_0 - t$	w
0.1	54.06	5.27	0.03	0.00
1	95.14	16.66	0.34	0.00
4	132.45	33.31	1.37	0.01
12	170.16	57.63	4.11	0.09
20	190.00	74.33	6.87	0.24
30	206.52	90.93	10.32	0.53
50	227.74	117.10	17.26	1.48
70	241.44	138.20	24.26	2.91
100	255.08	164.54	34.86	5.94
140	266.34	193.62	49.17	11.65
200	275.39	229.39	71.06	23.85
260	279.22	259.06	93.46	40.50
309	280.00	280.00	112.15	57.46
360	279.32	299.49	131.99	78.45
400	278.02	313.23	147.82	97.37

Energy Balance

The analysis of failure raises the two fundamental questions:

a) What was the speed of the vehicle bringing about the accident?

b) In the given case, what a minimum vehicle speed should have been required to cause the collapse of the bridge?

In connection with question a), no concrete answer can be given. The reason for it is the initial kinetic energy of the hauled vehicle having mass 'm' and speed 'v':

$$E_{\min} = \frac{mv^2}{2},$$

from which speed 'v' could be re-calculated, cannot be determined. With this in view, one must know that kinetic energy is 'absorbed' by the steel structure at the moment of impact, i. e. the mass colliding against the structure performs a non-recoverable amount of work during slowing-down, inasmuch it will cause a permanent deformation, fracture or heating of the steel elements. However, in this case this amount of work cannot be re-calculated from the condition of the wrecks, because the deformations, fractures and displacements resulted not only from the force exerted by the impact of the vehicle but also from the collapse of the structure rendered unstable by the effect of impact, as well as from its constrained motion. It

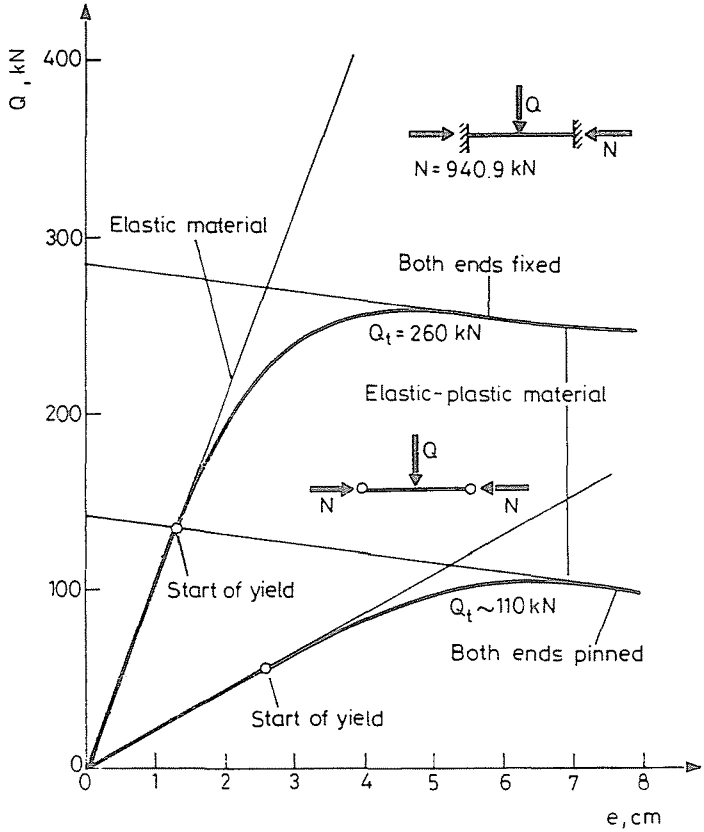

Fig. 7.

could have been well occurred, e. g. that the vehicle would have already stopped after a stopping distance of 1 m at a structure similar to that dealt with above but supported steadily on a scaffold as opposed to the case examined where the stopping distance of 1 m was impossible because the instability discussed in point 4.2. would occur already after a distance

$$u = 0.580\,\mathrm{m}\,,$$

and the chords — independently from whether the vehicle has any more kinetic energy — will be contracted and let the vehicle run farther (the distance covered by the vehicle will be increased).

The problem outlined above can also be illustrated on a very simple model. In *Fig. 8* a hinged column loaded by force of gravity 'N' of a constant magnitude can be seen, which is going to be held vertically by a spring below it. The characteristic curve is shown in *Fig. 9* (it corresponds to the characteristic curve of an ideally elastic-plastic material model used commonly with the dimensioning of steel structures). If a horizontal force of magnitude 'F' is acting on the top of the column and the spring resistance is sufficiently great, then the column rotated by angle κ will be restored into its state of equilibrium. In such a case — with every data assumed as known except of force 'F' — the value of force 'F' can be re-calculated by means of angle κ representing the motion of the column (the problem is unmistakable). If subsequently force 'F' is increased gradually on, then at a critical force 'F_{cr}' and a hinge moment M_{pl} and angle κ_{cr} belonging to it, the column will be getting into motion (becomes unstable), and 'falls down' horizontally ($\kappa=90°$). Of course, similarly a rotation by $\kappa=90°$ will be the final result with each force:

$$F \geq F_{\mathrm{cr}}\,,$$

from which it can be inferred that there is an infinite number of forces 'F' by the effect of which the column can fall down horizontally. Thus, in case $\kappa=90°$ is observed — if force 'F' bringing about the final state is unknown for some reason — the actual value of 'F' cannot be re-calculated (the problem is an undeterminate one). In a more simple case, the estimation of the force effect can be initiated from the measure of deterioration (permanent deformation, fracture) suffered by the structure rendered unstable. However, with the bridge examined here, it was impossible to evaluate reliably the damage suffered due to the fall-down of the bearing support.

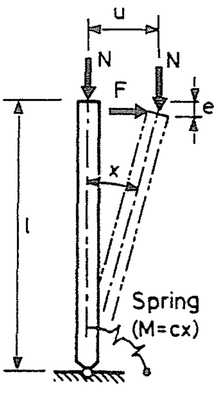

Fig. 8.

From the foregoing it follows that in the given case answer could be given only for case b).

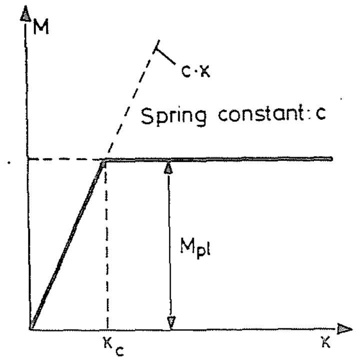

Fig. 9.

The impact bringing about the collapse can be examined by the model shown in *Fig. 10.* Since this model disregards the development of plastic hinge (the joints of the trussed girders of vertical plane are assumed to be hinged ones — as it is accepted in the practice of statics), the results approximate the reality from below. Due to it, it is reasonable to take into account an increasing factor of 1.5 with the minimum speed yielded.

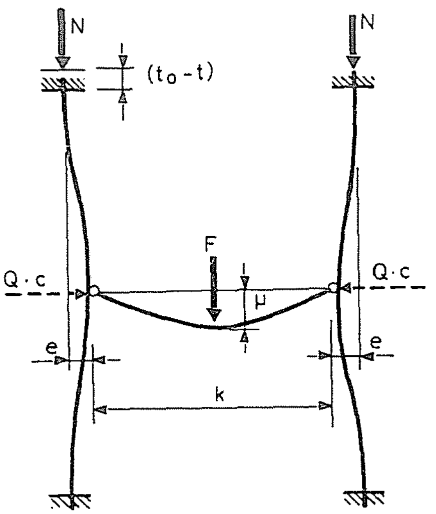

Fig. 10.

Otherwise, on the basis of those mentioned in the foregoing, the analysis of the unstable condition can take place by reckoning with the following data (See also *Figs. 5–7*):

$$\begin{aligned}
Q_t &= 260\,\mathrm{kN} \\
m &= \mu \cdot m_0 = 1.15 \cdot 19460 = 22380\,\mathrm{kg} \\
N &= 941\,\mathrm{kN} \\
u &= 0.58\,\mathrm{m} \\
e &= 0.06\,\mathrm{m} \\
(t_0 - t) &= 0.001\,\mathrm{m} \\
k &= 5.88\,\mathrm{m}
\end{aligned}$$

The kinetic energy of the hauled vehicles prior to impact:

$$E_{\mathrm{kin}} = \frac{mv^2}{2} = \frac{22380}{2} \cdot v^2 .$$

The approximate value of tensile force Q_t acting on the chord is:

$$Q_t \cong 260 = \frac{Fk}{4u} = \frac{F \cdot 5.88}{4 \cdot 0.5} .$$

Hence:

$$F = \frac{260 \cdot 4 \cdot 0.58}{5.88} = 103\,\mathrm{kN} .$$

Assuming that the heat output at the impact can be neglected, the change of kinetic energy E_{kin} will be equal to work L_{ext} performed by the external forces. Since the state is examined when the impact is just bringing about the unstable state, the following equation will hold:

$$\Delta E_{\mathrm{kin}} = E_{\mathrm{kin}} - 0 = E_{\mathrm{kin}} = L_{\mathrm{ext}} .$$

At the same time, the work performed in the system by the external forces will be:

$$L_{\mathrm{ext}} = \int N \mathrm{d}s + \int F \mathrm{d}u = 2 \cdot 941 \cdot 0.001 + \frac{3}{4} \cdot 103 \cdot 0.58 = 47.0\,\mathrm{kNm} .$$

Consequently, the minimum speed bringing about the instability will be:

$$v_{\min} = 1.5\sqrt{\frac{2E_{\mathrm{kin}}}{m}} = 1.5\sqrt{\frac{2 \cdot 47.0}{22.38}} = 3.01\,\mathrm{m/sec} \approx 11\,\mathrm{km/h} .$$

Consequently, the calculations show that the vehicle with a trailer was proceeding along the bridge at least at a speed of 12 km/h when the jib of the excavator got caught in the upper transverse bracing.

Conclusions

Before World War II, the trussed steel bridges were constructed with main girders of varying height in most cases. With such structures — if they were of troughttype — upper wind brace could often be mounted only in the middle section of the opening. This layout involves the danger that the vehicles having a load higher than the permissible one will deteriorate the chord in compression of the main girder in the middle section of the opening, which, in turn, — according to the experiences and analyses — can result in the collapse of the bridge even at a slow running speed (small impact force). With this taken into consideration, the following statements can be made:

- The mentioned structural layout is extremely dangerous, consequently it should not be applied to the new bridges.
- In front of the old bridges constructed with the structural layout mentioned above, railings of a steel structure (portal frames) should be mounted which will prevent the vehicles higher than permitted from driving on the bridge.

Address:

P. PLATTHY
Department of Steel Structures
The Technical University
H–1521, Budapest, Hungary

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