FULL-SCALE TESTS OF TEMPORARY STEEL (FOOT)BRIDGES

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ABSTRACT

The paper is focused on the problems of tests of steel structures intended for the temporary bridges and footbridges. On authors’ workplace, full-scale tests of existing railway temporary steel bridge and recently developed temporary steel footbridges have been performed. For the experimental verification of the railway temporary steel bridge in the real size the effective testing method non-requiring the fixed testing equipment anchored to the ground has been applied. In the case of newly developed temporary steel footbridges, the traditional loading methodology using the ballast load has been applied. The span of tested railway temporary bridge was 18 m and the spans of tested prototypes of newly developed temporary footbridges were 18 m and 36 m. The results of experimental verifications are the important information about the behaviour of real structures giving data of actual stresses, stiffness, deformations including the influence of individual structural parts and components on these, and can provide the backgrounds for the verification and correction of the calculating and evaluating procedures based on the usual analytical methods and numerical models.

Keywords: full-scale tests, steel, temporary bridge, footbridge, prototype.

INTRODUCTION

In the period of last three years, on authors’ workplace the full-scale testing of the existing railway temporary steel bridge and newly developed temporary steel footbridges has been performed. For the experimental verification of the railway temporary steel bridge in the real size the effective testing method non-requiring the fixed testing equipment anchored to the ground has been applied. In the case of newly developed temporary steel footbridges the traditional loading methodology using the ballast load has been applied. The span of the tested railway temporary bridge was 18 m, the spans of the tested prototypes of two types of newly developed footbridges were 18 m and 36 m. The results obtained from these experimental verifications are the information about the actual behaviour of real structures giving the data of the actual stresses, stiffness and interaction of individual structural parts and components and can provide the backgrounds for the verification and correction of the calculation procedures based on the usual analytical methods and numerical models. Within the framework of this experimental programme, before full-scale testing realization, the tests of the details of footbridges have been performed (Karmazínová, 2013b, Štrba, 2013a, b), i.e. static tests and mainly fatigue tests of selected exposed details of main structural members.

FULL-SCALE STATIC TESTS OF STEEL TEMPORARY BRIDGE “ŽBM 30”

The object of the theoretical and experimental investigation is the special railway bridge “ŽBM 30”, developed by railway army in the past and utilizable as the temporary bridge for the substitution of bridge structures with the spans up to 30 m. This system with web-plate
The test specimen was represented by the particular structural composition of the railway temporary bridge system of “ŽBM 30” for two main girders of the span of \( L = 18 \) m. The main girder consists of three segments – two trapezoidal ended segments with the length of 6 m and one rectangular middle segment also with the length of 6 m. The welded I-section of the main girder has the height of 1 800 mm in the span middle and 900 mm in the supports; the axis distance of main girders is 1 512 mm. The transverse stiffness of both girders is given by eight (in total) transverse bracings – four web-plate bracings displaced in the girder end parts and four truss bracings displaced in the girder middle part. The illustrations of the actual composition of the test specimen are in Fig. 1.

**Fig. 1 - Set-up of main steel load-carrying structure of “ŽBM” temporary bridge for the span of 18 m: a) view to main girders and supports represented by the members of “PIŽMO” system; b) truss transverse bracings**

**Realization of loading tests**

For loading tests realization the general loading principle without the need of any fixedly anchored loading frame, i.e. without the necessity of the testing equipment anchorage, has been utilized (Melcher, 1997). The system is composed of the tested girder and seated girder, that the tested girder is supported by the seated girder in the end supports. Across the upper edge of the tested girder and across the lower edge of the seated girders the cross-girders are lead; to the cross-girders the ties rods going besides of the tested girder are anchored. The ties are lead through hollow hydraulic cylinders, by which loading actions are generated; due to this effect the ties rods are tighten and they transfer the load to the tested girder. The general loading principle is evident from the scheme in Fig. 2. The loading system is in the equilibrium state. The seated girder must have the sufficient flexural stiffness and load-carrying capacity to be able to resist the introduced loading actions and not to have oversize deformations. The seated girder is supported by the end supports put on the ground to obtain the space enough under the seated girder for the cross-girders. Between the tested and seated girder there is the gap for free deformation of both main components of the testing system.

In the area of the FIRESTA Inc. Company in Brno – Modřice the load-carrying structure of “ŽBM” bridge for the span of 18 m has been assembled. For the erection the portal crane of load-bearing capacity of 60 tons has been applied. The arrangement of testing equipment based on the loading principle described above is shown in Fig. 3. For testing equipment (seated girders, cross-girders, ties) the members of various bridge systems for provisional purposes have been utilized.
The test specimen has been loaded introducing loading actions together to both main girders (test T1) and also introducing loading actions through one main girder (test T2), aimed to investigate the degree of interaction of both main girders and to verify the effect of transverse bracings from the viewpoint of the transverse distribution of loading. The test load has been derived from the values of the operating loading actions actual in the practice usage for provisional purposes and it has been limited by the condition, that no failure or damage should not occurred, because of the ability to use the temporary bridge in the next time. Based on that one, the test loading has been determined as continuous uniform load of 50 kNm\(^{-1}\) for one main girder. Because the loading method utilized in given conditions does not allow the uniform load application, loading actions have been realized applying two forces introduced to the main girders in the positions of hollow transverse stiffness.

In the case of test T1 tie rods have been lead outside the whole loading system to introduce the load together to both main girders. In the test T2 tie rods have been lead outside one main girder to introduce the load to this one girder only. The load has been introduced as the forces generating by hydraulic jacks: in the case of test T1 the cylinder forces V1 have been applied to the girder N1 and cylinder forces V2 have been applied to the girder N2 (Fig. 4); in the test T2 the cylinder forces V1 have been applied to the girder N1, while the girder N2 has not been directly loaded (Fig. 4).

It is possible to expect, that during the initial load cases when the load is low, the displacement of supports and connections occur at first, thus the loading process has been performed by 3 loading cycles including in common 39 load cases. The first loading cycle started by zero force values (load case 1) and the load has been increased step by step up to
the maximum force value (load case 17). In the second phase of the first cycle the sequential offloading has been applied to the force value of 75 kN (load case 21). The second loading cycle also included two phases, increasing from 21 to 25 load cases and decreasing from 25 to 29 load cases. The third loading cycle included the increasing phase from 33 to 39 load cases and the decreasing phase from 33 to 39 load cases that means to quite offload of the structure. The scheme of loading regime is drawn in Fig. 4.

![Fig. 4 - Loading forces in cylinders: test T1 and T2 (left), scheme of loading regime (right)](image)

**Results of loading tests**

The measurement methodology and the selection of observed quantities have been subordinated to the requirements of the co-operating company. Measuring points on the tested structure have been selected considering these requirements; during loading tests the vertical deflections and stresses of main girders have been monitored in these points. Measuring points have been located to the cross-sections with extreme expected deflections and stresses. Potentiometers measuring deflections have been located in the mid-span, in the segments connections and closed to the supports. Five sensors have been located in the same points of both girders (Fig. 5), to compare both girders deflections.

In the case of loading both girders (test T1), the deflections in corresponding points were nearly the same. It is evident, that the load is transferred to both girders practically identically, so both girders have been loaded equally. For the illustration, measured deflections are drawn in the graphs in Fig. 6: deflections of the girder N1 in the points P1–P5 for test T1 and also deflections of both girders N1 and N2 in the points P1–P10 for test T2 are drawn in Fig. 6. From the graphs in Fig. 6 it is obvious, that deflections of the girders N1, N2 have been significantly different in the case of load introducing to one girder only, even the signs of the deflections of girders N1, N2 were inverse i.e. the girder N2 moved up. Maximum deflections of the girder N1 in the case of test T2 were by less than 10 % smaller than deflections of the girders in the case of test T1. It indicates that the interaction of main girders realized by transverse bracings is practically negligible. If the load has been transferred through one girder only, the loading effect to next girder does not show almost at all and the girder directly loaded introduces the significant load part.
The stresses of the structure have been determined helping strain gauges and recalculating
strains to stresses. The gauges have been located in the points with expected extreme stresses,
i.e. in the mid-span, in the segment connections and in the cross-section change on the
beginning of the haunch. The strain gauges have been located on the lower surface of the
bottom flange: three gauges across flange width in the middle section and one gauge on
flange surface in other sections. The scheme of the location of gauges T1 to T16 on both
girders is drawn in Fig. 7. In the graphs in Fig. 8 the normal stresses in the girders N1 and N2
are drawn for the test T2, for the illustration. The stress distribution is drawn in the measured
points along the span for selected load cases.

Fig. 5 - Location of deflection sensors on lower flange surface (view in plan): girder N1
(upper), girder N2 (lower)

Fig. 6 - Load – deflection “F – w” diagrams: test T1 – girder N1 (left), test T2 – girders N1 and N2 (right)

Fig. 7 Location of strain gauges on lower flange surface (view from above): girder N1 (upper), girder N2 (lower)
As expected, from the graphs it is evident, that the stresses in the connections are considerably larger than the stresses in the mid-span, which is caused by the significantly smaller cross-section in the connection of the middle and support structural segments. Also from the figures it is evident, that in the case of the load introducing to one girder only, the load effect to the next girder does not show practically at all and the significant load part is introduced by directly loaded girder. It again confirms the obtained fact, that the degree of the interaction of both girders through the transverse bracing is minimal, so the transverse introduction of the load is not practically employed.

### Numerical analysis

The comparison of experimental results with the theoretical analysis and the investigation of the actual behaviour of structure tested in comparison with theoretical assumptions was one of the aims of the solution. The numerical model (see Fig. 9) has been created to determine deflections of main girders and stresses in the important places for their subsequent comparison with the experimental results and conclusions for the next verification of the numerical model.
For the numerical modelling, FEM analysis has been used. The calculation was performed for the selected load cases applied during loading tests T1 and T2 on the girder N1. With regards to the solution needs the outputs have been namely oriented to the values of deflections and stresses at the important points, where the strain gauges have been installed during the test.

Comparison of experimental and numerical results

The deflections of the main girders obtained from the loading test, as well as the deflections calculated using the numerical model are listed in Table 1 for the tests T1 and T2 for the selected load cases. From the results mentioned in Table 1 it is evident, that the differences between theoretical and experimental values are maximally 10 %.

The normal stresses obtained by the gauge measurement from the loading test, in comparison with comparison with the stresses numerically calculated the stresses numerically calculated are listed in Table 2 for the tests T1 and T2 for the selected load cases. From the results mentioned in Table 2 it is seen, that in some cases the theoretical values are lower than the values measured, while the difference does not exceed about 20 %, in the individual cases only.

<table>
<thead>
<tr>
<th>Force $V'_1$ [kN]</th>
<th>Test T1</th>
<th>Test T2</th>
<th>Numerical analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>P2</td>
<td>P3</td>
<td>P2</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>150</td>
<td>25.1</td>
<td>28.0</td>
<td>23.8</td>
</tr>
<tr>
<td>300</td>
<td>45.9</td>
<td>51.7</td>
<td>42.4</td>
</tr>
<tr>
<td>380</td>
<td>55.1</td>
<td>62.4</td>
<td>50.9</td>
</tr>
</tbody>
</table>
The stiffness of the main girder is going from the structure deflections, which do not exceed the value given of $L / 300$ as the basic value for the deflection limit of the usual structural members. Because the investigated structure is intended for temporary bridges, it is possible to take the value of $L / 300$ as satisfied from the viewpoint of the structure stiffness and serviceability.

<table>
<thead>
<tr>
<th>Force $V_1$ [kN]</th>
<th>Normal stresses $\sigma$ [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test T1</td>
</tr>
<tr>
<td></td>
<td>T2</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>150</td>
<td>167.9</td>
</tr>
<tr>
<td>300</td>
<td>299.0</td>
</tr>
<tr>
<td>380</td>
<td>377.5</td>
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The stresses in any measured points of the main girder do not exceed the yield strength of applied steel given in the producer sources. Maximum stresses for maximum loads reach about 76 % of the yield strength and the structure has sufficient load-carrying capacity reserve. As expected, the larger normal stresses occur in the connections of the end parts and middle part, which is influenced by the smaller cross-sections. But nor here the stresses are not larger than 76 % of the yield strength.

The results also show that the degree of interaction of girders is negligible if the load is introduced through one girder only. Thus, it is practically impossible to take into account transverse bracings effect to the load transfer to both girders. In such case, almost the complete loading effect is introduced by directly loaded girder, what shall be consistently respected in the practical usage.

FULL-SCALE STATIC TESTS OF STEEL TEMPORARY FOOTBRIDGES

The need of the development of new temporary footbridges has been, for instance, caused by the traffic conditions and safety situation at the construction sites. Especially in the case of socially important construction works, such as large bridges for example, the operation and organization at a construction site also requires the insurance of the pedestrian traffic. Also in the case of floods, the necessity of the usage of temporary bridge structures for the pedestrian and bicycle traffic is non-discussable. These two reasons gave the initiative to develop the new type of temporary footbridge. The following two variants of the footbridge differ not only in the span and self-weight, but also in the structural configuration (Karmazínová, 2013a, Karmazínová et al., 2013c) and member’s cross-sections.

Structural systems of developed steel temporary footbridges

The short footbridge (see Fig. 10) is composed as the structure with lower deck and two truss main girders with the axial distance of 2 360 mm and theoretical height of 1 390 mm. The length of one segment of the main girder is 1 000 mm. The geometrical system of the main girder is composed of the parallel straight chords, diagonal and vertical members; the diagonals in every third segment are crossed. The spatial rigidity of the footbridge is ensured by longitudinal horizontal truss bracing.
The chords of truss main girders are rectangular tubes. The load-carrying structure of the footbridge deck is composed of cross and longitudinal girders. Cross girders with the axial distance of 3.0 m are placed between lower chords of the main girders. Longitudinal girders with the axial distance of 0.72 m and 0.82 m, respectively, are placed between cross girders.

The stability of the compression upper chords of main girders is ensured by the system of the transverse frames in the distance of 3.0 m. Transverse frames are composed of cross girders and verticals and they have a sufficient flexural stiffness, to support the compression upper chords of main girders in horizontal direction. The footbridge structure has one longitudinal bracing only, in horizontal level of the footbridge deck. The longitudinal bracing is structured as the truss plane girder of rhombic system. The chords of the longitudinal bracing are represented by the lower chords of main girders; the verticals are cross girders; the diagonals have rectangular cross-sections.

The long footbridge (see Fig. 10) is composed as the structure with lower deck and two truss main girders with the axial distance of 2 360 mm and theoretical height of 2 670 mm. The length of one segment of the main girder is 3 000 mm. The geometrical system of the main girder is composed of the parallel straight chords, diagonal and vertical members; the diagonals are crossed. The spatial rigidity of the footbridge is ensured by longitudinal horizontal truss bracings. The chords of truss main girders, as well as vertical members, are rectangular tubes; the diagonals have circular cross-sections. The load-carrying structure of the footbridge deck is composed of cross and longitudinal girders. Cross girders with the axial distance of 3.0 m are placed between lower chords of the main girders. Longitudinal girders with the axial distance of 0.72 m and 0.82 m, respectively, are placed between cross girders. Both cross girders, and longitudinal girders are rectangular tubes.

Two longitudinal bracings are used: the lower bracing at horizontal level of the footbridge deck and the upper one at the level of the main girder upper chords. Both bracings are structured as the truss plane girders of rhombic system. The chords of the lower longitudinal bracing are represented by lower chords of main girders; the verticals are cross girders; the diagonals are circular tubes. The chords of the upper longitudinal bracing are represented by upper chords of main girders; verticals are rectangular tubes and diagonals are circular cross-sections.

Both structures are divided into assembly parts, which are being connected by assembly joints. They are located on upper and lower chords of main girders in the places of vertical members in the distances of 3.0 m, in accordance with main girder assembly segments. Before the production of footbridges prototypes, two segments of the footbridge (see Fig. 11) have been produced, to verify the production tolerances and connections clearances, from the viewpoint of the production procedure and surface protection, for which the heat galvanization has been chosen, finally.
Realization of loading tests

To verify the actual behaviour of the developed footbridges as a whole the loading tests of the entire structures have been realized (Karmazínová, 2013a, Karmazínová et al., 2013c). The main aim of loading tests was to verify both structural systems from the viewpoint of serviceability limit state, and in parallel, from the viewpoint of ultimate limit state. Within the limit state of serviceability the stiffness of structural systems as a whole has been verified measuring the deflections of the whole structure subjected to usual operating loads. Within the ultimate limit state the stresses in the chords of main girders have been monitored measuring the relative deformations using strain gauges.
For loading tests the structure with the span of 18 m in the case of short footbridge and the structure with the span of 36 m in the case of long footbridge have been assembled. The real configuration of footbridge prototypes manufactured is shown in Fig. 12. After the structures assembly (see above), measuring devices and indicators have been installed – see illustration in Figs. 13 and 14.

The load of the short footbridge has been represented by steel profiles as a substitution for variable load service load usual for normal traffic conditions. The weight of each beam was 942 kg and the total number of them was 9 pieces. The loads have been introduced to the structure in two load cases (LC) – see Fig. 15. The first load case LC-1 was symmetrical; the steel beams have been placed uniformly along the span of footbridge. The total weight of beams was 8 478 kg, that the load was about of 2.35 kNm acting on both main girders. The second one LC-2 was asymmetrical; the beams have been placed on the one side of footbridge. In this case the load value of one main girder was about 3.95 kNm and the load of the second one was about 0.8 kNm approximately.
The loads of the long footbridge have been represented by packages of building materials, weight of each one was 1 000 kg. Testing loads have been introduced to the structure gradually (see Fig. 16), so that its value, when the load was over the whole footbridge length, was 3.33 kNm⁻², which correspond with the usual operating loads.
Results of loading tests

During the loading process, the deflections \( w \) and normal stresses \( \sigma \) have been monitored and measured continuously to obtain their progress in dependence on time, which also means on load progress. Illustrations of progress of the deflections and normal stresses at the mid-length measured on the main girder chords are shown in Fig. 17 (short) and Fig. 18 (long).

In the case of short footbridge, the maximal deflection in the mid-length for service load case LC-2 is approximately equal to 40 mm. This value is less than the deflection limit given by the value of \( L / 300 = 18 \, 000 / 300 = 60 \) mm, so that it reaches about of 2/3 of this limit. The normal stresses in the main girder chords reach the maximal value about of 60 MPa that means safely under the nominal yield strength value, which is equal to 355 MPa (steel S 235).
In the case of long footbridge, the maximal deflection in the mid-length for service load LC-1 over the whole footbridge length is approximately equal to 55 mm. This value is much less than the deflection limit given by the value of \( L / 300 = 36000 / 300 = 120 \) mm, so that it reaches less than about of 1/2 of this limit. The normal stresses in the main girder chords reach the maximal value about of 70 MPa that means safely under the nominal yield strength value, which is equal to 355 MPa (steel grade S 355).

CONCLUSION

The particular conclusions have been mentioned and discussed in the text above, individually for the static loading tests of steel temporary railway bridge, as well as for the static loading tests of newly developed steel temporary footbridges.

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