The development and testing of a new type of the temporary steel truss footbridge with closed cross-section

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***Abstract*—**In this paper there is some particular information about the development of a new sort of a temporary modular steel footbridge for pedestrians and cyclist, which was designed as a truss system with the deck below the supports and with the closed cross-section. Actually, this development was connected to the research project, which was one of the several ones from the recent period, where they all were focused on temporary steel footbridges design, developing and testing (including full-scale testing) and which were realized on our workplace in cooperation with research centres and companies. In this case the new footbridge has a span 18 - 36 m and it is divided into 3.0 m long assembly units. They were selected some its details for the loading tests and finally it was performed also a test of the prototype. Therefore, this paper brings the results of those experiments as well as some particular conclusions.

***Keywords*—**Loading tests, pin connections, repeated loading, steel footbridge, temporary footbridge, truss system.

I. Introduction

SEVERAL research projects, focused on steel temporary bridges design and testing, were in the recent period realized on the authors' workplace, which is Brno University of Technology and its part AdMaS Research Centre at Faculty of Civil Engineering in cooperation with the Technology Agency of the Czech Republic as well as with the Vladimír Fišer Company.

It can be mentioned for example the realization of the full-scale testing and load-bearing capacity verification of the steel temporary railway bridge with welded web-plate main girders and with the span of 18.0 m (see [1]), which is a part of so called ŽBM 30 System, originally designed as a temporary military bridge.

However, this paper is focused on another research project, where the main aim was the design of the new steel footbridge. In fact, they were several main reasons for a development of a new type of a temporary footbridge. First, it was the providing of a simply self-supporting pathway for pedestrians and cyclists in case of reconstructions, new building sites as well as in case of some natural disasters. In all this cases it is usually needed some fast replacement of previous damaged or unusable structure. Moreover, the other reason was also the possibility of a creating or a replacement of a permanent footbridge. And finally it can be mentioned, that there is actually in the Czech Republic a serious shortage of this type of bridge constructions in general.

Therefore, they were at the beginning defined several requirements for this structure, which were the reliability and safety as well as the condition, that this temporary construction has to be easy to assemble and its single parts and components have to be easily storable. Besides, the design had to satisfy all condition given in European Standards.

As the results they have been designed and developed two steel temporary truss footbridges [2]–[7]. The first one was so-called "short" or "small" footbridge and it was created as the simply supported beam for 3.0 to 18.0 m span with the open cross-section, where the stability of its compressed upper chords is ensured only by floor beams and verticals (which together create the rigid U-frame). More specific information about the loading tests and the development of this construction can be found in. The second one (that is more specifically described in this paper) was so-called "long" or "large" footbridge, whereas it has closed cross-section and it was intended for 18 to 36 m span [8]. The example for 30.0 m span is in Fig. 1.

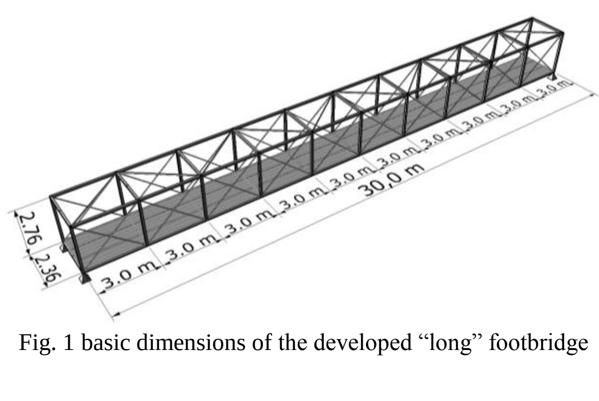

Fig. 1 basic dimensions of the developed "long" footbridge

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II. Geometry and Cross-Sections of the Footbridge

The developed “long” steel footbridge has been designed as a simply supported steel truss beam with one span and with the deck below the support. It is perpendicular and it has a straight longitudinal axis. The load-bearing system consists of two parallel main truss girders of height 2.76 m, where the distance between them is 2.36 m.

This footbridge is as well composed of separate 3.0 m long assembly units, where each unit is represented by one single truss panel. The cross-section of the footbridge is closed with the top and bottom lateral bracings made of diagonals and verticals. The geometry and dimensions are shown in Fig. 1. The steel grade was S355.

In Fig. 2 there is one of the FEM models, which was used in case of the footbridge optimization in the phase of its design and development. In this specific model it was used always only one single diagonal member in each truss panel of parallel main girders. Then, after the optimization they were finally used two crossed diagonals in each truss panel as it is obvious in Fig. 1 as well as in Fig. 3, where it is presented the full-scale prototype of described footbridge (which was used for loading tests).

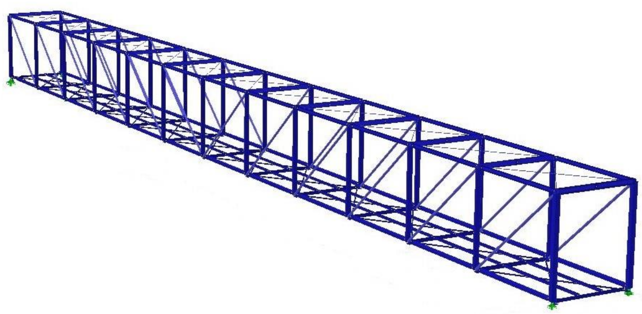

Fig. 2 the original FEM model of the footbridge

Fig. 3 the footbridge prototype used for full-scale testing

For upper and lower chords of main girders as well as for verticals they have been used hot rolled rectangular box cross-sections. Their individual dimensions are following: top chords TR 4HR 100×100×4, bottom chords and usual verticals TR OBD 140×80×4, then verticals at the end of the footbridge TR 4HR 140×140×5. For the diagonals of main girders they have been used solid circular tension rods KR 30.

The deck is designed classically and consists of floor beams and stringers. Both of them are also made of rectangular box cross-sections (TR OBD 80×40×4). The distance between floor beams is the same as the distance of assembly units, it means 3.0 m. Next, the distance between stringers is 0.72 m respectively 0.82 m.

For the verticals in top bracings they have been used profiles TR 4HR 140×140×5 and finally, for the diagonals in bracings (top and bottom), they have been used circular tension rods and tubes. The completed footbridge prototype is shown in Fig. 3.

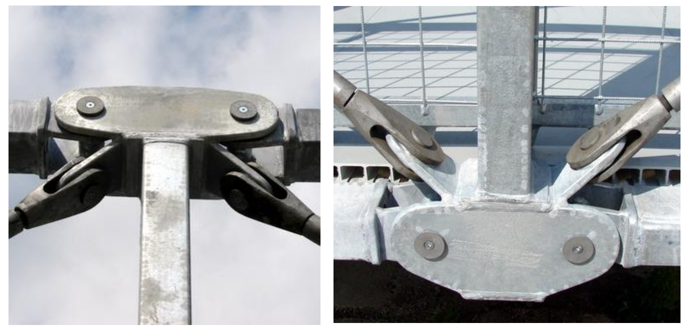

Fig. 4 the pin connections of the upper (left) and lower (right) chords

The connections of all footbridge members are designed and made as welded. The only exception they are the assembly joints, where they have been used pins. As an examples there are in Fig. 4 shown pin connections of bottom and top chord as well as of the diagonals of bracings.

For the top layer of the deck of this temporary footbridge they were used the composite panels with the longitudinal stiffeners (this part was researched separately and it is not a subject of the described research).

III. The Loading Tests of the Footbridge

In case of the described “long” steel temporary footbridge development they were at the beginning selected important construction details which were the upper and lower joints of the steel truss main girders, namely for the verification of their load-carrying capacity as well as for the obtaining information about their actual behaviour under the loading. In this case they have been firstly realized two tests with use of the static axial force and then several loading tests using the cyclic force.

A. The equipment for static and cyclic loading tests

The realization and initialization of the loading forces have been ensured by use of the hydraulic cylinder anchored to the steel loading frame together with the strain gauge load cell (eventually with the induction position sensors if needed).

The capacity of this cylinder is 1000 kN and it allows both static as well as cyclic loading. The cylinder is controlled by the control equipment with its appropriate software.

Some more specific information about described loading equipment can be found in [2]. Some examples of mentioned loading arrangement are shown in Figs. 5 and 6.

B. Static loading tests

In case of the load-carrying capacity verification of selected details under the static loading they have been realized 2 tests of the upper respectively lower chord connections – the first one with the tension force (see Fig. 6 left) and the second one with the compression force (in Fig. 6 right and Fig. 7). In case of compression force it had to be used the additional support for the stabilization of the specimen under loading.

Both of those experiments were stopped after it was reached the force of about N = 400 kN while the specimens still remained without any failure. The results are in Tab. 1 and then in the graph in Fig. 5 they are the force-to-deflection relationships.

Table 1 the recapitulation of tests with use of static axial force

Test number	Type of loading	Loading force N_{max} [kN]	Mode of failure
1	Tension	397.1	none
2	Compression	-402.6	none

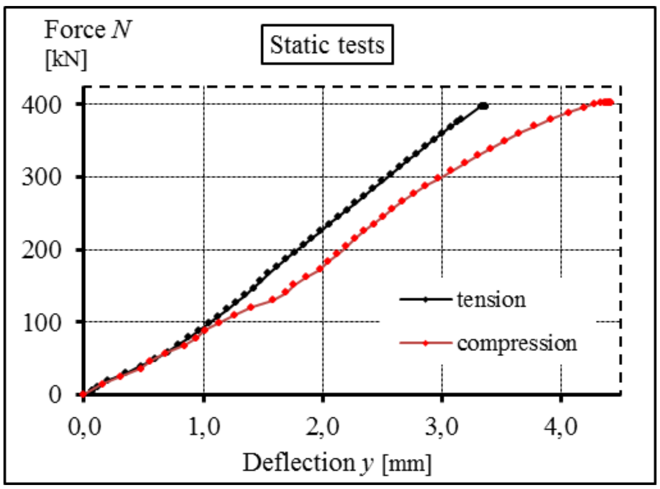

Fig. 5 the force-to-deflection relationships in case of static tests

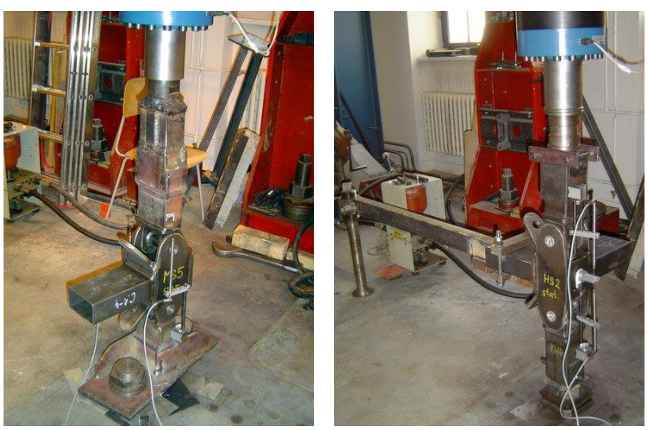

Fig. 6 the illustration of static loading tests with the tension (left) and compression axial force (right)

Fig. 7 the additional support of the specimen in case of the static loading test with use of a compression axial force

C. Cyclic loading tests – general information

While the static tests of connections were performed only to confirm the assumption of the sufficient load-bearing capacity of selected details, the cyclic tests were intended to verify that these details are efficiently usable in case of the repeated loading with specific amplitudes of tension and compression forces.

D. Cyclic loading tests – the realization

In order to create the tension and compression cyclic forces it was used the hydraulic equipment (which was described in part A of this chapter) with controlled force. In case of each test the loading forces were first increasing linearly and after reaching the previously selected mean value N_m they were subsequently changed into maximum and minimum depending on chosen loading amplitude ΔN. Some more information about the initialization phase and about the process of cyclic loading itself are described in [2] and [3]. The frequency of all cyclic loading tests was $f = 5.0$ Hz.

Altogether, they were performed 5 tests of the joints of the "long" temporary footbridge main truss girders with use of cyclic loading. For the first 3 of them it was used the repeated tension force and then, for the next 2 tests, the compression force.

In case of tension loading the amplitudes of forces ΔN were successively set at 119.0, 97.0 and 75.0 kN. Next, in case of compression loading they were selected as 148.0 and 99.0 kN (as it is described in Table 2).

Table 2 the recapitulation of the selected forces amplitudes

Test number	Type of loading	N_m [kN]	N_{min} [kN]	N_{max} [kN]	Loading amplitude ΔN [kN]
1	Tension force	84.5	25	144	119
2		73.5	25	122	97
3		62.5	25	100	75
1	Compression force	86.5	25	148	123
2		74.5	25	124	99

During all the cyclic tests they were continually tested individual components of joints, which, after their failure or after achieving usually more than two (respectively three) million cycles, were always exchanged for the another part of connection. In this manner they could be tested several specimens per each experiment on the selected level of loading force amplitude ΔN. This way they have been finally tested altogether 39 specimens.

In case of loading tests of the connections with use of the cyclic axial forces all the specimens have been divided and identified by the location in the joint respectively chord as follows: "**MS**" is the **middle part** of joint of the **lower chord** with the splice plates and holes for the pins, "**VD**" is the end of the **lower chord** of the main truss girder with the splice plates together with the holes for the pins, "**MH**" is the **middle part** of joint of the **upper chord** with the splice plates and holes for the pins and finally "**VH**" is the end of the **upper chord** of the main truss girder with the splice plates and with the holes for the pins. Some of these markings, written on the test specimens, are shown in the Figs. 6 - 9). Except these parts they have been tested also the pins and they have been marked according to joint in which they have been used.

As an example of typical failure modes of tested specimens they are some pictures in Figs. 8 and 9.

Fig. 8 the illustration of the failed specimens in case of tension and compression joints under repeated loading (failed splice plate of the middle part – top and bottom left, failed splice plate of the end part of lower chord – top right, failed pin – bottom right)

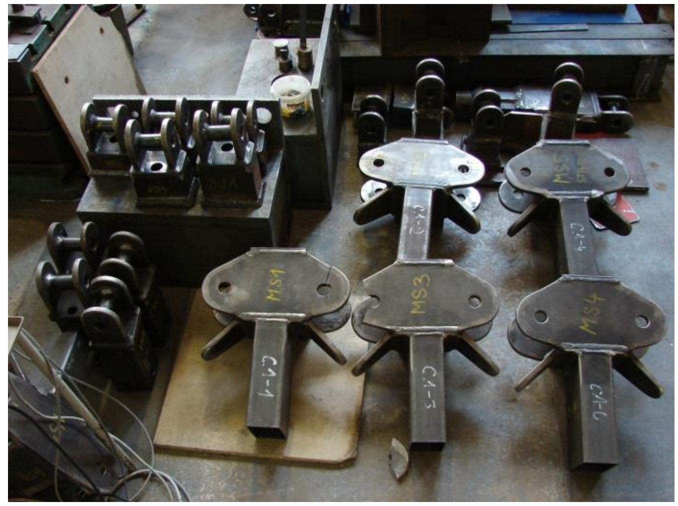

Fig. 9 the failed specimens of the temporary footbridge joints

E. Cyclic loading tests – the results

In the graph in Fig. 10 they are shown the results (failures of all tested specimens) in case of using of cyclic tension force.

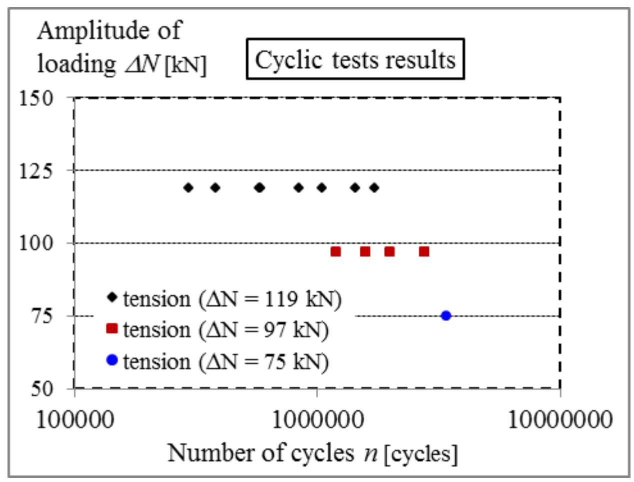

Fig. 10 the results in case of cyclic tests using tension force

Next, in Tables 3 - 8 there are the results obtained during the cycling loading tests (with use of tension and compression forces) in case of the pin connections of main truss girder. It should be mentioned as well, that they have not appeared any weld failures in connections during the testing.

Table 3 results of tension cyclic tests in case of "MS" members

Test	Specimen	ΔN [kN]	n [cycles]	Failure mode
1	MS1 – a	119	1 727 000	none
	MS1 – b			fracture
	MS2 – a		278 000	none
	MS2 – b			none
2	MS3 – a	97	2 782 000	fracture
	MS3 – b			none
3	MS4 – a	75	3 404 000	none
	MS4 – b			none

Table 4 results of cyclic tension tests in case of "VD" members

Test	Specimen	ΔN [kN]	n [cycles]	Failure mode
1	VD1	119	2 005 000	none
	VD2		1 431 000	fracture
	VD3		574 000	fatigue crack
2	VD4	97	1 579 000	fracture and fatigue crack
	VD5			fatigue crack
	VD6		1 203 000	fatigue crack
	VD7			none
3	VD8	75	3404000	fracture
	VD9			none

Table 5 results of cyclic tension tests in case of pins

Test	Specimen	ΔN [kN]	n [cycles]	Failure mode
1	1	119	845 000	fracture
	2		1 049 000	fracture
	3		586 000	fracture
	4		382 000	fracture
	5		574 000	fracture
	6		296 000	fracture
	7		278 000	none
2	8	97	2 782 000	fracture
	9		1 993 000	fracture
	10		789 000	none
3	11	75	3 404 000	none
	12			none

Table 6 results of compression cyclic tests in case of "MH" members

Test	Specimen	ΔN [kN]	n [cycles]	Failure mode
1	MH1 – a	123	982 000	none
	MH1 – b		982 000	none
2	MH2 – a	99	3 000 000	none
	MH2 – b		3 000 000	none

Table 7 results of compression cyclic tests in case of "VH" members

Test	Specimen	ΔN [kN]	n [cycles]	Failure mode
1	VH1	123	982 000	none
	VH2			none
2	VH3	99	3 000 000	none
	VH4			none

Table 8 results of compression cyclic tests in case of pins

Test	Specimen	ΔN [kN]	n [cycles]	Failure mode
1	1c	123	982 000	fracture
	2c			none
2	3c	99	3 000 000	none
	4c			none

F. Full-Scale test of the prototype of the footbridge

After all static and cyclic loading tests performed for individual parts (joints) of described steel temporary footbridge, it was performed also a full-scale load test of the prototype of this footbridge.

Fig. 11 shows two connected 3.0 m long assembly units, which are prepared for galvanization in the assembly hall.

Fig. 11 prepared assembly units before (left) and after a galvanization

In case of the prototype loading test it was decided to use the maximal span, it means 36.0 m (consist of 12 assembly units). During this test they were measured the deflections and the values of a stress of the footbridge members.

In Fig. 12 it is shown the scheme of positions of the potentiometric deformation sensors (the total number of them was 8). Then, the Fig. 13 shows the positions of strain gauges (the total number was 16).

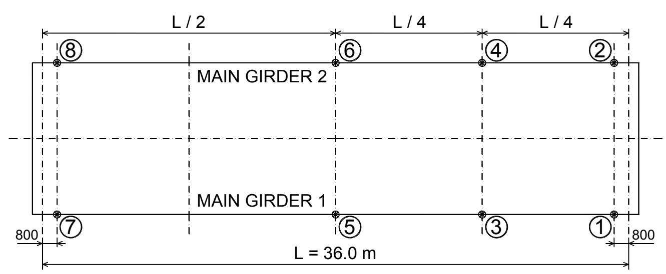

Fig. 12 the positions of the potentiometric sensors of deformations

The strain gauges were placed in the middle and in the quarter of the span of footbridge on the upper as well as on the lower chords of the main girders, whereas in case of the main girder number 1 they were placed even on the top and bottom fibres (SG1, SG2, SG3, SG4, SG7, SG8, SG9, SG10) and in case of the main girder number 2 they were used only for the top fibres (SG5 and SG11) respectively for the bottom fibres (SG6 and SG12). Except that, the strain gauges were used for the end diagonals in case of both main girders (SG13, SG14, SG15 and SG16).

The example of both potentiometric sensor as well as strain gauge is on photographs in Fig. 14.

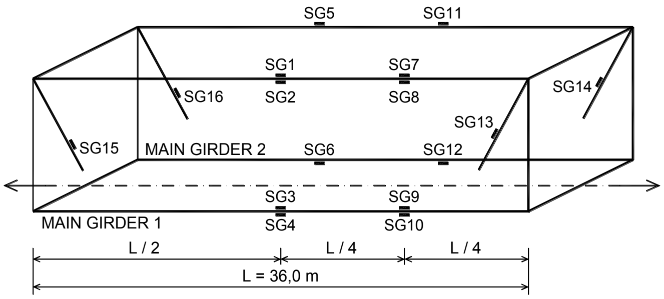

Fig. 13 the positions of the strain gauges

Fig. 14 an example of used potentiometric sensor of deformations (left) and an example of used strain gauge (right)

Except the self-weight of the footbridge it was used also the alternative to create variable (uniformly distributed) load. For this purpose they were used the pallets (containing some specific insulation material), where the weight was 1.0 ton per pallet. They were placed in the middle of each 3.0 m long assembly unit (in longitudinal as well as in transverse direction of the footbridge – see Fig. 15). It means it was created the uniformly distributed load of 1.67 kN/m for each main truss girder. In Fig. 16 it is shown the distributing of the pallets.

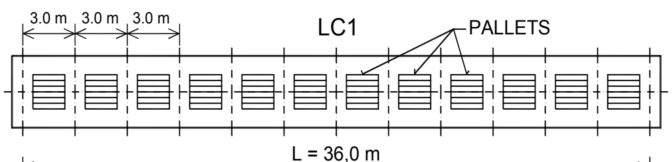

Fig. 15 the scheme of the arrangement of pallets (1 pallet = 1000 kg)

Fig. 16 the illustration of the process of distributing of the pallets

Fig. 17 the final state in case of the uniformly distributed load

The final state of described load case (marked as LC1 in Fig. 15 and in the next graphs and tables, see below) with the pallets is shown in Fig. 17.

G. The results of the Full-Scale test

In Fig. 18 they are the deflection-to-time relationships in case of described load case with uniformly distributed load for all measured points according to layout in Fig. 12).

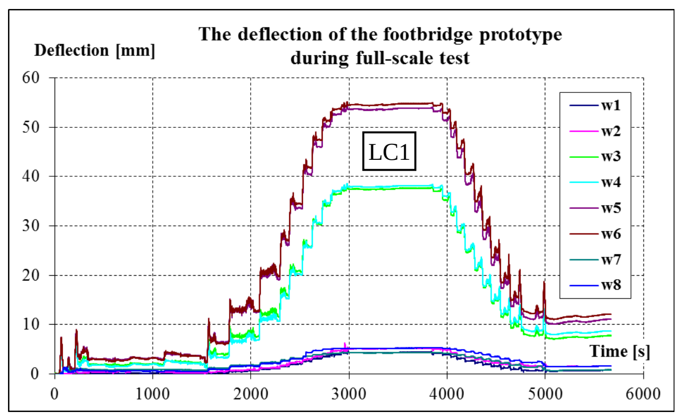

Fig. 18 the deflection-to-time relationships of the footbridge prototype in case of distributed load

From this graph they are obvious all the phases of the loading process. The individual values of the maximal deflections in case of uniformly distributed load are described in Table 9 and next as well in Fig. 19.

The maximum value of a deflection occurred (as expected) in the middle of the span of the truss footbridge prototype. The concrete value was about 54 - 55 mm (see Table 9). The limit value of the deflection (relevant for temporary steel footbridges in general) is $L/150$, where "L" value is a span of a footbridge. In this case it means that the limit was 240 mm. Thus, the deflection reached about 25 % of the allowed value for this type of construction.

However, it has to be mentioned, that the deflections were measured only in case of variable load. Then, together with the deflections from the self-weight the maximum value was reaching about 50 % of the allowed value (the relatively big deflection from self-weight is to the clearance in pin joints).

Table 9 the values of deflections in case of distributed load (LC1)

The deflection in all positions according to Fig. 12 in case of uniformly distributed load [mm]							
w1	**w2**	**w3**	**w4**	**w5**	**w6**	**w7**	**w8**
4,40	5,14	37,68	38,18	53,88	54,84	4,28	5,23

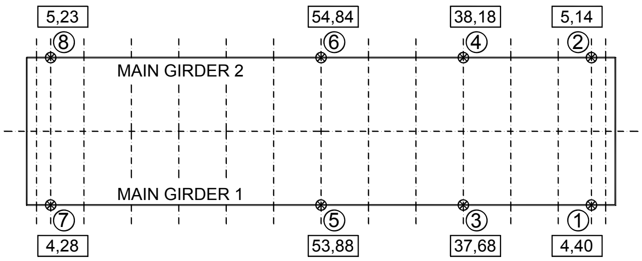

Fig. 19 the values of the deflection in case of distributed load

In Figs. 20 - 22 they are the graphs of the relationships of a stress in dependence on time in case of the footbridge prototype test for all the positions of the strain gauges SG1 to SG16 defined on Fig. 13.

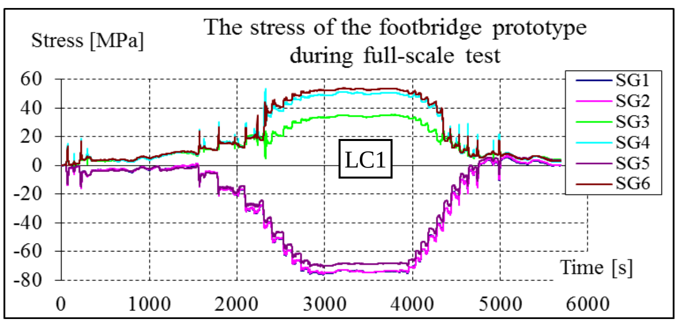

Fig. 20 the values of the stress (for strain gauges SG1 to SG6)

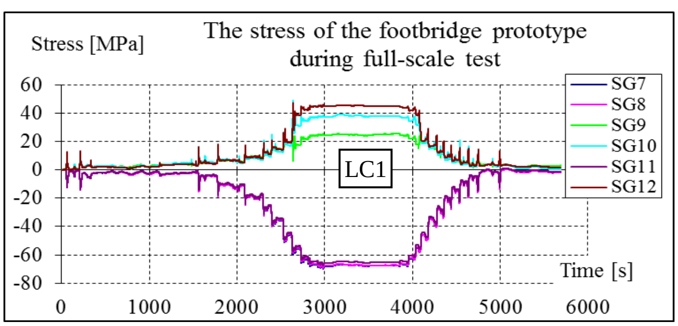

Fig. 21 the values of the stress (for strain gauges SG7 to SG12)

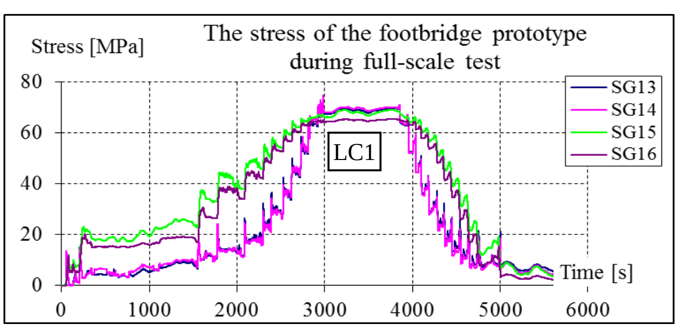

Fig. 22 the values of the stress (for strain gauges SG13 to SG16)

In the Table 10 they are the final results (it means the maximum values of stress) for each strain gauge according to Fig. 13. The values of the stress reach about 15 % of the capacity of used steel.

Table 10 the values of deflections in case of distributed load (LC1)

The stress in all positions according to Fig. 13 in case of uniformly distributed load (LC1) [MPa]							
SG1	**SG2**	**SG3**	**SG4**	**SG5**	**SG6**	**SG7**	**SG8**
-75,0	-75,0	35,3	51,4	-69,5	53,9	-68,2	-68,0
SG9	**SG10**	**SG11**	**SG12**	**SG13**	**SG14**	**SG15**	**SG16**
25,4	39,0	-65,9	45,6	69,6	70,1	69,1	65,4

IV. Conclusion

Some specific particular conclusions have been already mentioned in the previous text above via described test results. The described temporary steel truss footbridge was designed and developed according to the normative rules, which are given by European Standards [9]–[14] in case of the satisfaction of the condition for ultimate limit state (ULS) and serviceability limit state (SLS) as well as for the requests about enough space for the pedestrians, etc.

Next, after tests with use of static tension and compression forces it was confirmed expected load-bearing capacity. Depending on these performed tests they were selected several specimens, which were tested in case of cyclic loading.

Then, based on previous experiences with cyclic loading tests of structures and members [15], the obtained results (it means the values of the loading amplitudes and the total numbers of cycles for all specimens) have been processed by the help of the methodology of design assisted by testing given in the Annex “D” of the *Eurocode 1* [9] to determine and verify the ultimate load-bearing capacity of selected critical important details (the assembly connections) in case of repeated tension and compression loading.

This testing methodology was used also for the another temporary steel footbridge of span up to 18 m [2]–[7] and it is planned to use it as well in case of other prepared steel bridges (including the temporary railway bridges, where the parameters are firstly designed depending on cyclic load).

Finally it was performed the full-scale test of developed temporary steel truss footbridge by using a prototype of 36 m span to confirm the sufficient capacity (especially in case of the deflection and stress).

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