On the problems of testing methodology used in case of the temporary steel through truss footbridge development

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Abstract**—**This paper brings some information about a performed research project in which a testing methodology was used in case of development of a new type of temporary steel truss footbridge for pedestrians and cyclists. At first, the mentioned structure was designed according European standards using finite elements method software and then the point of research was to experimentally verify the actual behaviour of the footbridge in the event of real loading. For that purpose they were firstly performed tests on selected members and details using tension and compression static force. According the obtained results it was used cyclic force for some parts as well. Finally, there was performed a test of full-scale prototype as well. In this paper there are some particular results and illustrative photos of the research presented.

Keywords**—**Actual behaviour, joints, loading test, steel structure, temporary footbridge.

I. Introduction

RECENTLY, within the framework of research projects made by our Institute of Metal and Timber Structures in co-operation with the Vladimír Fišer company and with the Technology Agency of the Czech Republic, it was designed a new type of temporary steel truss footbridge for pedestrians and cyclists [1].

The entire idea of the project was to develop a new type of simply temporary structure, which could effectively create as well as replace a self-supporting way in case of some events like reconstructions, building sites or natural disasters (floods, for example). This temporary footbridge could be also used as a permanent footbridge if needed (actually it happens very often in the Czech Republic in general). They were defined a few most important requirements, too. At first it was a safety and reliable design. Then, the structure parts had to be easy to storage and assembly.

Within the framework of the project solution it was used the testing methodology, which included a realization of several loading tests of selected details to get information about actual behaviour of them and to make an efficient design [2], [3].

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II. Footbridge Description

The described temporary footbridge is designed according to normative rules [4]–[9] as a simply supported steel truss structure with single span of 15.0 m length and with the deck below the support (it means the type was so-called through truss bridge).

The footbridge is straight and its centre line is direct. The load-bearing structure is open and it consists of two main parallel truss girders of 1.39 m height, whereas the distance between them is 2.36 m. The structure is made of five 3.0 m long assembly units where the each unit contains three panels of 1.0 m length (see Fig. 1 and 2). For the whole structure it is used a steel with ultimate strength of 355 MPa.

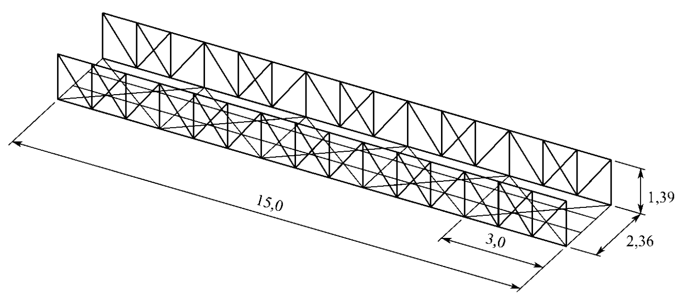

Fig. 1 temporary footbridge geometry

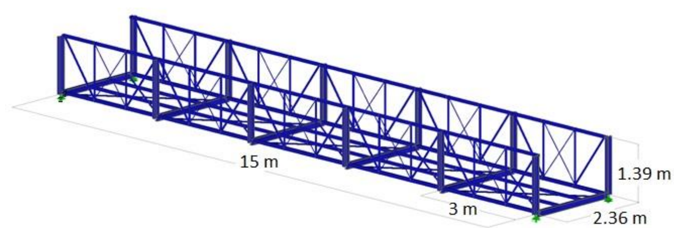

Fig. 2 temporary footbridge FEM model

The stability in longitudinal direction is provided by lateral bracing (diagonals) made of plates 40 mm × 4mm in every second truss panel. Stability of compression top chords of open footbridge is ensured by the U-frames made of floor beams and verticals per 3.0 m, which is the length of an assembly part. Field splices are made of pins (see Fig. 3).

Fig. 3 the detail of upper chord joint made as pin connection

For upper chords, lower chords as well as for diagonals they has been used hot-rolled rectangular hollow sections (top chord 100 mm × 60 mm × 4 mm, bottom chord 80 mm × 40 mm × 4 mm and diagonals 40 mm × 40 mm × 3.2 mm). Cross-sections of U-frames they are hot-rolled IPE140 and verticals are made of 40 mm × 40 mm × 3.2 mm box sections. The deck consists of floor beams and stringers, whereas for both of them there are used 80 mm × 40 mm × 4 mm box sections.

III. Loading Tests

In case of footbridge development and research they have been realized 4 loading tests using static (tension and compression) force and 6 loading tests using tension cyclic force. On the Fig. 4 they are shown some prepared specimens of investigated joints.

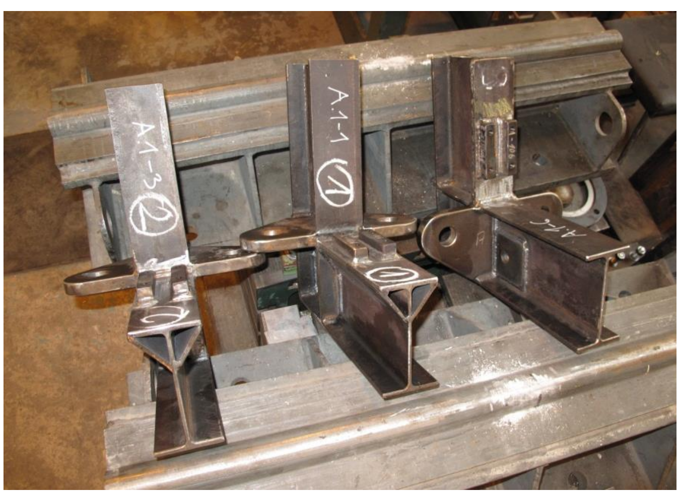

Fig. 4 prepared specimens of tension joint

Finally, it was performed an experiment of a footbridge prototype in situ [10]. Particular information about loading tests and their results are described below.

A. *The description of load tests equipment arrangement*

For the realization of loading tests it was used hydraulic servo-cylinder anchored to steel loading frame. For the measuring of forces it was used load cell connected directly into cylinder. The whole load arrangement was operated by control equipment.

The intensity of loading forces was adjusted through the computer with the relevant software along with the recording of this data altogether with the corresponding data of deflection obtained by measuring centre.

The scheme of equipment including each particular part mentioned above is on Fig. 5. For some pictures of described loading system see Fig. 6.

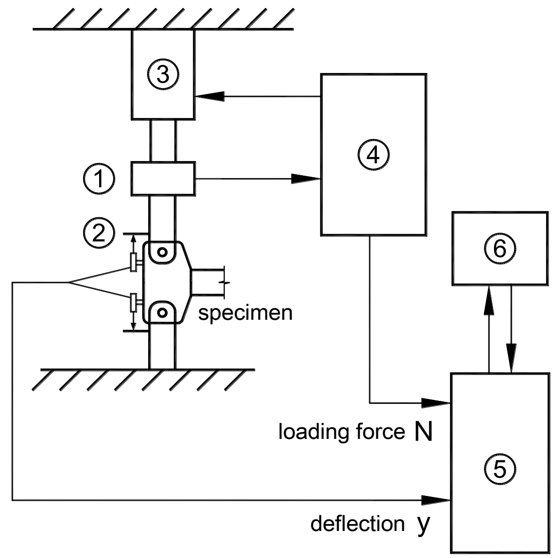

Fig. 5 scheme of the load test equipment arrangement

The description of equipment parts (according to Fig. 5):

1) Strain gauge load cell U5/100, by HBM.
2) Induction position sensor WA-T 50 mm, by HBM
3) Servo-cylinder AG 400-100, by INOVA Prague.
4) Control equipment EU2000D, by INOVA Prague.
5) Measuring center MGC plus, by HBM.
6) Computer software Catman Easy, by HBM.

Fig. 6 steel loading frame used in case of testing of specimens

B. The realization of static loading tests

They were selected two specimens of upper chord tension joint and two specimens of top chord compression joint for tests with static loading force to verify an ultimate capacity of these parts. In case of static loading tests it was deflection measured, too.

On Fig. 7 they are photos of the experiments with tension force and also there is an illustration of test with compression force on Fig. 8 where the additional support was used for the stabilization of specimen under compression.

Fig. 7 illustration of static tension loading tests

Fig. 8 illustration of static compression loading tests

C. The realization of cyclic loading tests

For the tests with cyclic force (depending on results of the static ones) [11], [12] they were selected only tension specimens of lower chord joint, which are designed as pin connections. Some of specimens were simple, without equivalent deck and vertical members (Fig. 9); the others were complete (see prepared specimens on Fig. 4).

Fig. 9 illustration of loading tests realization

The start-up phase of tension force initialization has been carried this way: in the beginning the force was linearly growing into starting mean value N_m and then the load have been subsequently changed into interval between maximum N_{max} and minimum N_{min} values of tension force depending on loading total amplitude ΔN. The value of N_{min} was selected as 50.0 kN and the value of N_{max} as 120.0 kN or 150.0 kN, respectively.

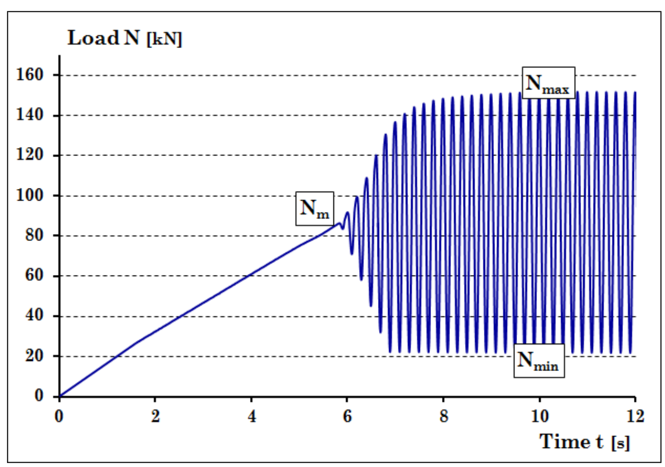

Fig. 10 typical example of the initial phase of cyclic loading during specimen testing

In control equipment of hydraulic servo-cylinder they were firstly adjusted two values. At first, the mean value of tension force N_m and then the tension force amplitude N_a to create the total forces amplitude ΔN, see (1).

$$\Delta N = N_m + N_a = N_{\max} - N_{\min}. \tag{1}$$

It was also selected the criterion of automatic shutoff for loading arrangement as the limit of total deflection, i.e. the relative position of single specimen parts. This value was set out as value of 50 mm.

The characteristic (typical) course example of initial (start-up) phase of specimen loading using tension cyclic force is shown on Fig. 10.

From the point of view of measuring centre setup (it means in terms of tension force realization by hydraulic equipment) it was a loading with controlled force. The frequency of loading cycle f was selected as 5 Hz (with using sinus amplitude). The sampling rate of induction position sensors was preset to 100 Hz per channel.

D. The realization of the footbridge prototype test

At the end of the research project it was performed also a load test of full-scale prototype of described footbridge. On Fig. 11 they are two prepared 3.0 m long parts of this prototype just before hot-dip galvanization in assembly hall. Next, on Fig. 12 they are the same parts after galvanization and with finished footbridge deck.

Fig. 11 two prepared assembly units before galvanization

Fig. 12 part of footbridge prototype with finished deck

In case of this in situ load test it was decided to use even bigger span of the footbridge then it had been designed for. So that they were used 6 assembly units altogether, it means, the total span of the structure was 18.0 m.

During the test they were a deflection of the footbridge as well as a stress in steel members measured. On Fig. 13 it is the scheme of potentiometric displacement sensors layout (the total number of sensors was 8; half of them on the one hand).

Then, there are positions of strain gauges on Fig. 14 typified. They were in the middle and at the one end of the footbridge, whereas 8 of them were on main girder no. 1 and the rest on girder no. 2 (they were to the bottom chord, end diagonal and vertical fixed).

On Fig. 15 they are shown examples of used displacement sensors and strain gauges.

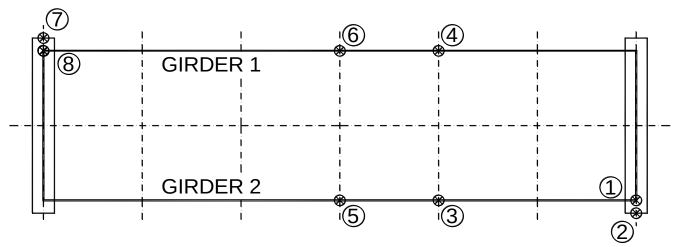

Fig. 13 layout of potentiometric displacement sensors

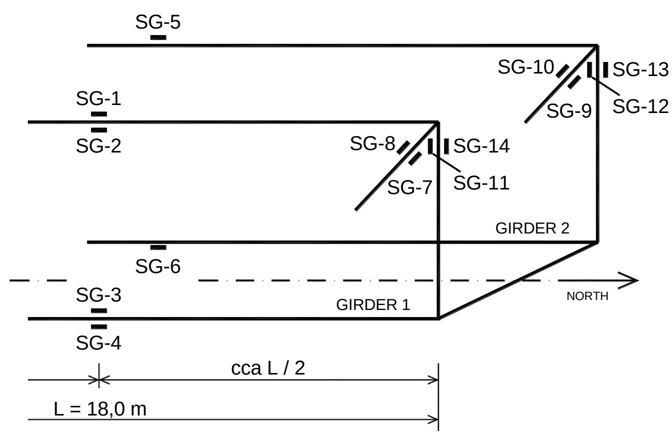

Fig. 14 layout of strain gauges

Fig. 15 an example of used displacement sensor and strain gauge

On Fig. 16 there is a prepared footbridge prototype just before the loading test.

Fig. 16 footbridge prototype of 18.0 m span prepared to load test

Except self-weight of the structure (it means dead load) they were used also steel beams as a substitution for variable (service) load. A weight of each one was 942 kg and the total number of them was 9 pieces. They were selected two load cases (LC) in the event of prototype load test.

The first one (LC-1) was symmetrical; the steel beams were placed uniformly along the span of footbridge. The total weight of beams was 8478 kg, which means there was variable load about 2.35 kN/m acting on both main girders.

The second one (LC-2) was asymmetrical; when the beams were placed on the one side of footbridge. In this case one main girder was loaded by value of 3.95 kN/m and the second one by 0.8 kN/m, approximately (see Fig. 17-19).

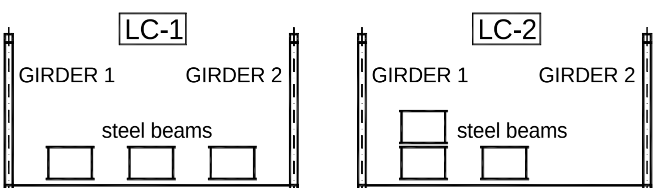

Fig. 17 arrangement of steel beams in case of used load cases

Fig. 18 illustration of load case 2 (LC-2)

Fig. 19 illustration of load case 2 (LC-2)

IV. Test Results

A. Results of load tests with static force

They were performed 4 loading tests using static force (two using tension and two using compression load). The results are described in Table 1 and 2.

In the event of static tests with tension force, the failure of pin and weld occurred (see Fig. 20).

Fig. 20 weld failure in case of static tension load test

Table 1 Results of tests using static tension force

Test	**Specimen**	**F_{max}** [kN]	**Mode of failure**
1	**TS1**	391,1	weld failure
2	**TS2**	399,5	pin failure

Unfortunately, in case of the first load test with compression force the additional support failed, so that the test had to be canceled obviously.

On the contrary, after second compression test the specimen stayed without any failure reaching value of 400 kN force (which was the limit of load equipment). There was only a slightly bearing failure of pins as well as of the holes for the pins (see Fig. 21).

The relationships of a deflection to the load dependences for all static load tests are illustrated in graphs on Fig. 22 and 23.

Fig. 21 specimen after failed support (top on the left) and slightly bearing failure of pin and hole

Table 2 Results of tests using static compression force

Test	**Specimen**	**F_{max}** [kN]	**Mode of failure**
1	**CD1**	216,5	test failed
2	**CD2**	399,6	none

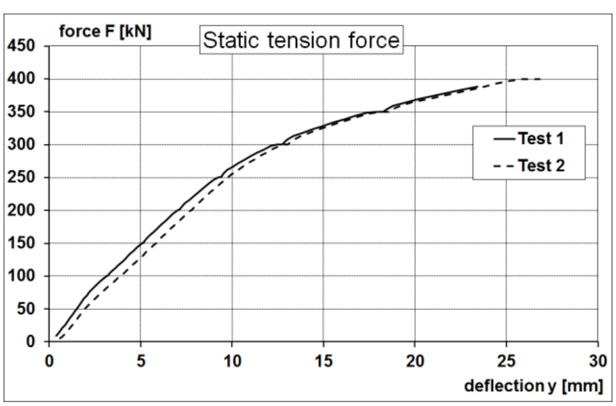

Fig. 22 load-deflection relationships in case of tension force

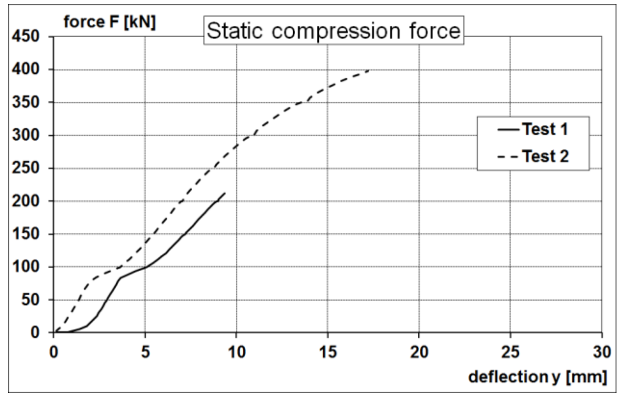

Fig. 23 load-deflection relationships in case of compression force

B. Results of load tests with cyclic force

They have been performed 6 tests of the main truss girder tension lower chord joints using cyclic tensile force. For the first test it was set the total amplitude ΔN as the value of 100 kN (i.e. 50–150 kN interval). For the rest of them the ΔN was 70 kN (i.e. 50–120 kN).

From that, once there was no failure in specimen after more than 2 million cycles (then the test was ended). In the rest five cases they occurred weld failures; always at point of chord end, where the hollow cross-sections is welded to the end plate. Examples of these failures of specimens are represented by Fig. 24.

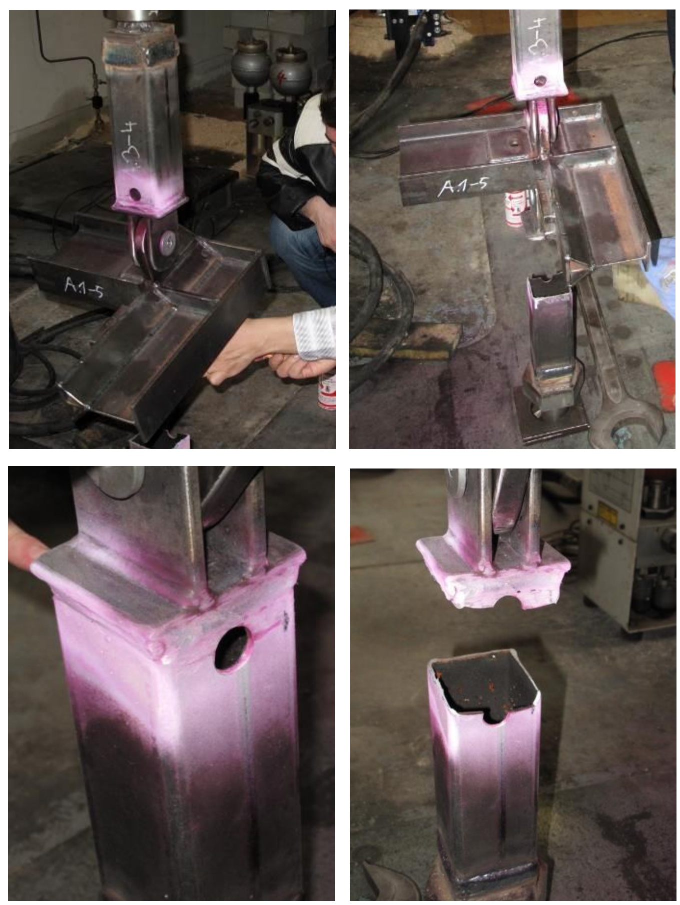

Fig. 24 examples of failure modes during cyclic tension loading tests in case of lower chord tension joint

Next, for the illustration, the Table 3 brings the summary of these experiments results (including numbers of cycles n).

Table 3 Results of cyclic loading tests of steel footbridge lower chord tension connection

Test	Specimen	Tension loading force		Loading amplitude	Number of cycles	Mode of failure
		N_{min}	N_{max}	ΔN	n	
		[kN]	[kN]	[kN]	[cycles]	
1	**TD1**	50	150	100	39500	weld f.
2	**TD2**	50	120	70	1336000	weld f.
3	**TD3**	50	120	70	842000	weld f.
4	**TD4**	50	120	70	1219500	weld f.
5	**TD5**	50	120	70	2074000	none
6	**TD6**	50	120	70	644 000	weld f.

C. Results of prototype load test in situ

On Fig. 25 they are the curves of deflection in dependence on time in case of both load cases for each measured point defined on Fig. 13.

They are obvious from these courses all the phases of loading process. First there was an increasing of deformation as the footbridge had been subsequently loaded by steel beams. Then the first load case (LC-1) occurred. After that the used steel beams had been repositioned at the one side of footbridge (see Fig. 17) to create the second load case (LC-2).

Next, the maximum values of deflection in each measured point 1-8 according to Fig. 13 are written in Table 4.

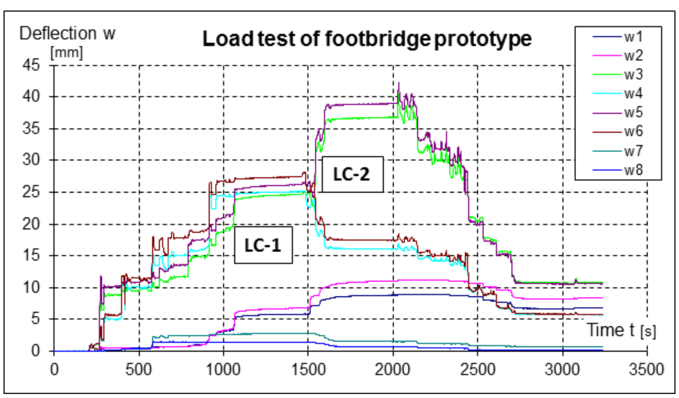

Fig. 25 deflection-time relationships in selected points 1-8 according to Fig. 13

Table 4 Values of deflections in case of both load cases

Load case	Deflection in points according to Fig. 13							
	w1	w2	w3	w4	w5	w6	w7	w8
	[mm]							
LC-1	5,78	6,77	24,70	25,10	26,20	27,50	2,77	1,41
LC-2	8,84	11,10	36,90	16,60	39,00	18,00	2,13	1,01

The maximum (unstable) value of deflection in the middle of steel footbridge span occurred as had been expected in case of second load case (the asymmetrical one). The value was 42.4 mm. The limit value of deflection is defined as L/150, (relevant for temporary footbridges) where L is a span of the bridge [9]. In this case the limit was 120 mm. It means the deflection of variable load reached 35% of this limit value.

On Fig. 26 and 27 they are the graphs which represent relationships of stress in dependence on time in case of the footbridge prototype test for all the given positions of strain gauges 1-14 defined by Fig. 14.

Then, in Table 5 they are the results (it means the maximum values of stress for both load cases) written for each position.

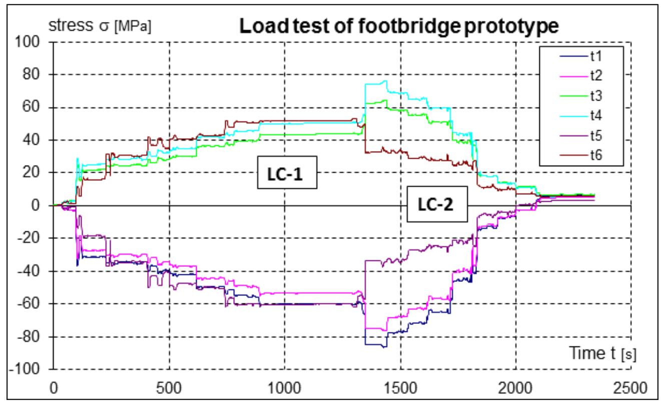

Fig. 26 stress-time relationships in measured points 1-6 (it means in the middle of footbridge) according to Fig. 14

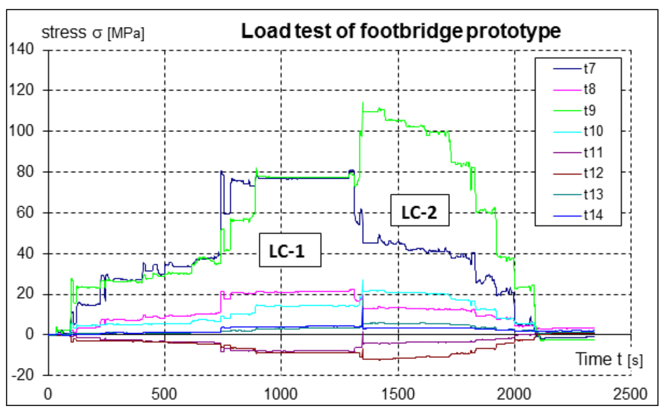

Fig. 27 stress-time relationships in measured points 7-14 (it means at the one end of footbridge) according to Fig. 14

Table 5 Values of stresses in case of both load cases

Load case	Stress in points according to Fig. 14						
	[MPa]						
	T1	**T2**	**T3**	**T4**	**T5**	**T6**	**T7**
LC-1	-60,3	-53,6	43,8	50,6	-60,5	52,0	77,0
LC-2	-77,9	-68,6	58,8	69,4	-35,1	34,2	46,3
-	**T8**	**T9**	**T10**	**T11**	**T12**	**T13**	**T14**
LC-1	21,4	77,8	14,3	-8,1	-8,9	3,2	4,3
LC-2	13,6	106,1	21,1	-4,3	-11,7	5,9	3,6

V. Conclusion

The described steel truss structure was designed according normative rules (in term of enough space for pedestrians, also of ultimate and serviceability limit states satisfying, etc.) [1]-[3]. Then, depending on tests using static compression and tension force they were selected details for experiments with cyclic loading.

On the basis of previous experiences with experiments with cyclic tension forces [13], [14], all the obtained test results (i.e. total amplitudes ΔN and numbers of cycles n) have been compared with known static values. For that purpose it has been used methodology of design assisted by testing according to Annex D of [4] to determine and verify the ultimate load-bearing capacity of those critical details in case of cyclic tension loading.

This testing methodology was successfully used also for the other temporary steel truss footbridge of 36.0 m long span (see Fig 28) and it is planned to use it in the future also for permanent steel truss footbridges and bridges. It can be used as well for different structure details, especially steel connections, where it is the possibility that they could be subjected to repeated tension as well as compression loading.

Fig. 28 load test of another newly developed steel footbridge of 36.0 m span, where the described testing methodology also was used

Finally, it will be used as well for development and efficient conceptual design of some fatigue details in case of research of advanced steel temporary railway bridges effective operating parameters, where the cyclic loading can occur continuously.

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References

[1] M. Karmazínová, “Structural design of newly developed temporary footbridge”, *Applied Mechanics and Materials*, Trans Tech Publications: Zurich, Vol. 405-408, 2013, pp.1623-1626, doi: 10.4028/www.scientific.net/AMM.405-408.1623. ISSN 1660-9336.

[2] M. Karmazínová, J. Melcher, M. Pilgr and M. Štrba, “Experimental verification and analysis of temporary bridge structure actual behaviour”, In *Proceedings of The Seventh International Conference on Advances in Steel Structures “Advances in Steel Structures”*, Southeast University Press, Nanjing, 2012, 2. 4. 2012, Volume 1, pp. 162-170, ISBN 978-7-5641-3400-6.

[3] M. Štrba, M. Karmazínová and P. Simon, “The testing methodology of new developed steel temporary truss footbridge details in case of cyclic loading”, In *Recent Advances in Civil and Mining Engineering – Proceedings of 4th European Conference of Civil Engineering (ECCIE'13)*, WSEAS Press, Antalya, Turkey, 8. – 10. 10. 2013, pp. 207-210, ISSN 2227-4588, ISBN 978-960-474-337-7.

[4] EN 1990 *Basis of Structural Design*, CEN Brussels, 2004.

[5] EN 1991-1-1 *Action on Structures* – Part 1-1: *General Actions – Densities, Self-weight, Imposed Loads for Buildings*, CEN Brussels, 2010.

[6] EN 1991-2 *Action on structures* – Part 2: *Traffic Loads on Bridges*, CEN Brussels, 2010.

[7] EN 1993-1-1 *Design of Steel Structures* – Part 1-1: *General Rules and Rules for Buildings*, CEN Brussels, 2006.

[8] EN 1993-1-8 *Design of Steel Structures* – Part 1-8: *Design of Joints*, CEN Brussels, 2010.

[9] EN 1993-2 *Design of Steel Structures* – Part 2: *Steel Bridges*, CEN Brussels, 2006.

[10] P. Simon, Š. Kameš, D. Weinstein, M. Karmazínová, J. Veselý and V. Salajka, “Development of temporary footbridge for pedestrian and cyclist traffic” (original in Czech language), In *Proceedings of the XV. Conference on Steel Structures 2013*, SEKURKON, Karlova Studánka 2013.

[11] M. Karmazínová and P. Simon, ”Fatigue tests of assembly joints of truss main girders of newly developed temporary footbridges”, *International Journal of Mechanics*, North Atlantic University Union, U.S.A., Vol. 7 (2013), Issue 4, pp. 475-483. ISSN 1998-4448.

[12] M. Karmazínová and P. Simon, “Experimental verification of fatigue resistance of assembly joints of truss main girder of temporary footbridge”, In *Proceedings of the 8th International Conference on Continuum Mechanics (CM ´13) “Recent Advances in Continuum Mechanics, Hydrology and Ecology”, “Energy, Environmental and Structural Engineering Series, No. 14”*, WSEAS Press: Rhodes Island 2013, pp. 80-85. ISSN 2227-4359, ISBN 978-960-474-313-1.

[13] M. Štrba and M. Karmazínová, “Actual behaviour and objective load-carrying capacity of tension steel expansion anchors to concrete, In *Procedia Engineering*, Elsevier, Žilina, Slovakia, 2012, 26. 9. 2012, Volume 40, pp. 440-444, ISSN 1877-7058.

[14] M. Štrba and M. Karmazínová, “On the problems of actual behaviour and load-carrying capacity of steel anchor bolts subjected to repeated tension loading”, In *Proceedings of 3rd European Conference of Civil Engineering (ECCIE'12) “Recent Advances in Engineering”*, WSEAS Press, Paris, France, 2012, pp. 273-276, ISSN 1877-7058, ISBN 978-1-61804-137-1.