

THE USE OF UP-TO DATE ANALYSES FOR THE TEMPORARY BRIDGES APPLICATION IN THE PRESENT

Richard HLINKA^{1,*}, Matus FARBAK¹, Jaroslav ODROBINAK¹

¹ Department of Structures and Bridges, Faculty of Civil Engineering, University of Zilina, Univerzitna 8215/1, 010 26 Zilina, Slovakia.

* corresponding author: richard.hlinka@uniza.sk

Abstract

Bridges are a critical element of the transport network. In case of neglected maintenance, war conflict or natural disaster, they may be damaged or destroyed. An alternative solution in the form of a detour is mostly feasible only in the case of ground communication. But even in this case, it can be a very difficult and lengthy solution. Therefore, temporary bridges are often used to replace the permanent bridge structure for a certain time. The history of temporary bridges is essentially as old as that of permanent bridges. The first patented temporary bridge was the Eifel Bridge, which had a span of up to 60.0 m in 1888. Before the Second World War, the most widespread structures were the German S-bridge with spans of up to 15.0 m, Herbert-brücke with a span of up to 25.0 m and the English Bailey bridge system. In the post-war era, the ŽM 60 temporary bridge was designed as a universal bridge structure, which was to serve to temporarily bridge obstacles of medium and long spans. As standard, it was to serve as a railway bridge with a span of up to 60.0 m and as a road bridge with a span of 84.0 m. The task of the study published in this paper is to approach the possibilities of deploying the ŽM 60 bridge at present in compliance with the requirements arising from currently applicable standards and regulations.

Keywords:

Temporary bridge;
Traffic load;
Stability;
Numerical model;
Load-carrying capacity.

1 Introduction

Temporary bridge structures were developed in the past mainly for military use. They were used for the relatively quick transfer of military equipment across obstacles, especially waterways [1], [2]. Modern warfare no longer assumes the use of these constructions in combat operations. Armies use temporary bridges more often to ensure the logistics of rear security or during humanitarian operations, where they ensure the supply of humanitarian equipment. Their biggest advantage is relatively quick assembly and almost immediate use [3], [4].

Using these military facilities is also becoming increasingly common in the civilian sector. They find their application in crises during natural disasters [5], [6], collapse of permanent bridges [7], etc. In these cases, temporary bridges are often used for a long time and sometimes replace permanent ones. It should be remembered that these structures were mostly designed in the last century and were not expected to be used as permanent bridges. The superstructure of these bridges was designed and verified using old design procedures and standards [8], [9]. With the development of methodologies for assessing bridge constructions with the boom in the use of computer technology in the design and assessment of structures, we are reaching a point where it would be appropriate to analyse these constructions in detail. It is necessary to focus not only on the possibilities of using already existing types of temporary bridges but also on analysing them to obtain knowledge that will be used in the design of new modern types of temporary bridges [10], [11], [12].

A modern temporary bridge has to meet several requirements. First of all, it should be multifunctional - it should be usable in both the military and civilian sectors, and it should be usable for road and rail transport. Other conditions include simple assembly and transport, high load-bearing capacity even for long spans, low-cost maintenance, and the possibility of simple repair. Existing

temporary bridges can be a source of inspiration [13], [14]. Most of these defined requirements are also met by the temporary bridge ŽM 60 [15], [16].

The ŽM 60 bridge was developed in the 1980s in the former Czechoslovakia. Its construction allows it to bridge obstacles with a span of up to 90.0 m. It can be used for road and rail transport. Currently, it is mainly used in crises and the construction of new bridges as a construction site bridge [17]. Only a few pieces of temporary bridges of type ŽM 60 were produced. In practice, we often encounter the replacement of original structural elements with various replacements that may not work correctly.

For the reasons stated, it was decided to subject this type of structure to a detailed analysis and determine the possibilities of general use of the ŽM 60 bridge. The analysis is focused on usability in the military but also in the civilian sector. Moreover, as already mentioned, the analysis results will be used not only in assessing the existing structure but also in potentially designing a new modern structure for the temporary bridge.

2 Description of the temporary bridge

The ŽM 60 bridge is a detachable steel truss system (Fig. 1) used for the fast bridging obstacles of medium and long spans. It is used in general crisis situations or emergency situations of the permanent bridge of railway and road infrastructure [5], [16]. The bridge consists of several parts, which are connected using bolts and pins. For the structural arrangement of the bridge, a regulation [15] is used, which describes the possibilities and requirements of the assembly.

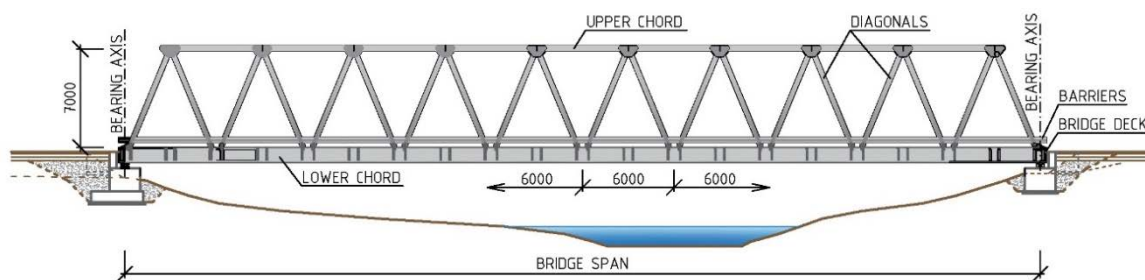


Fig. 1: Side view of the temporary bridge ŽM 60.

The bridge deck is a plate made of a 20 mm thick steel sheet with stiffeners. There are longitudinal ribs in the shape of rolled IPE in the place below the rails. Longitudinal closed ribs are also located on the bridge deck for transferring road traffic load. The lower bridge deck is discreetly connected by crossbeams to the main girders at a distance every 3.00 m.

The main truss girders have flexural rigid bottom chords. By default, the structure is designed as a Warren truss. The length of one section is 6.0 m. This length can be shortened to 3.0 m using a half-vertical section, if necessary. The lower chords are designed as a welded I-shaped beam with a height of 1.0 m. The web of the beam splits into a Y-shape at the flanges. The upper chord and diagonals are made of an H-shaped welded steel profile. They are lightened by circular holes. The connections of the chords and diagonals are constructed as bolt joints. The connection of the bottom chords is provided with forged ridge locks for the longitudinal connection of parts by seven-shear pin connections (Fig. 4).

The function of lower longitudinal bracing is performed by a slab bridge deck. The stability of the upper chords is ensured by the upper longitudinal bracing.

In terms of spatial arrangement, a bridge cross-section 4.8 m wide and 6.5 m high is created on the bridge.

The original design of the structure took into account the regulations [15] in force at that time. In the case of rail transport, the main girder up to a span of 54.0 m is designed for train load model "B" in the sense of [8]. For spans up to 60.0 m, train load model "C" and train load model "A" according to [8] are used. The bridge deck is suitable for loading by train load model "B" in the sense of [8].

Road traffic is defined by the load model of one vehicle with a total weight of 80.0 tonnes, then a system of forces with a value of 1040 kN and a uniform surface load with an intensity of 5.0 kN/m². The bridge is suitable for such loads up to a span of 84.0 m.

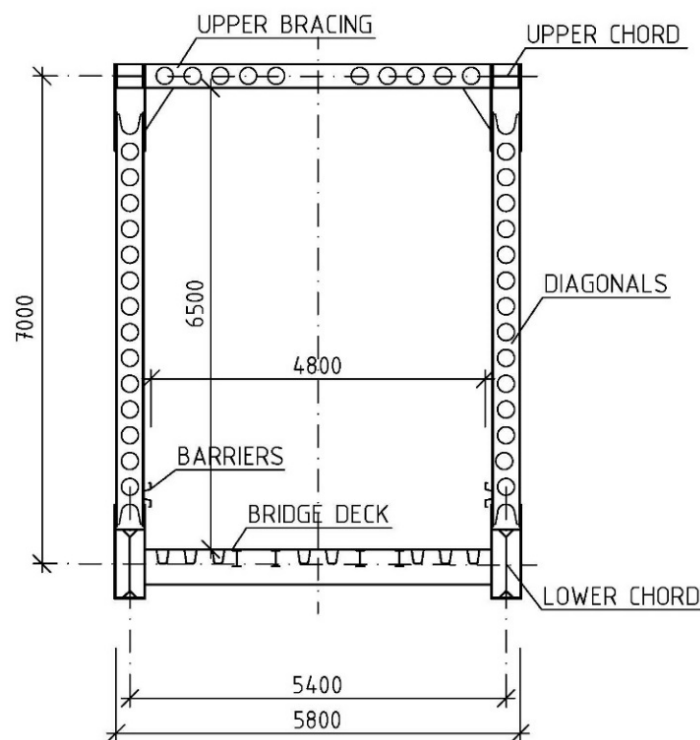


Fig. 2: Cross-section of the ŽM 60 bridge.

3 Numerical model

3.1 Description of the numerical model

Numerical models of the structure were created to find out the possibilities of using the temporary bridge ŽM 60 in individual situations (Fig. 3). A total of 11 models with different spans were developed. This varied from 30.0 to 90.0 m, with a modulus of 6.0 m. The model was created using the SCIA Engineer software [18], which works based on the finite element method.

The bridge deck plate was modelled by 2D elements with the longitudinal and transverse ribs (modelled as 1D beam elements). The effects of shear lag and plate buckling on members' stiffness were considered in the global analysis. The main girder (chords and diagonals) and upper longitudinal bracing were modelled using 1D beam elements. The final models take into account the actual dimensions of the structures and all eccentricities.

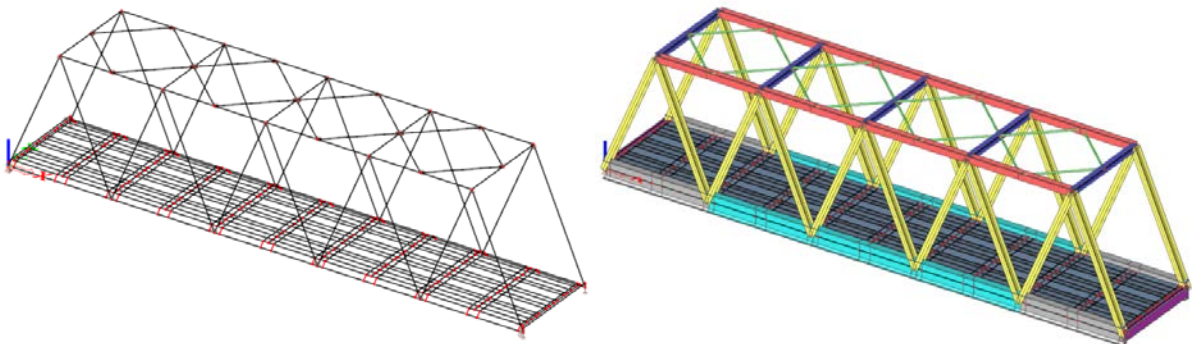


Fig. 3: FEM model of the temporary bridge.

The individual connections of the chords are considered to be hinged [19]. Special attention was paid to the joint of the lower chord of the main girder (Fig. 4). The resistance of the connection is discussed in a study [20] or [21] in detail. Based on the numerical analysis [22], it is possible to assume that the joint will safely transfer a force of 4,656.00 kN, which represents 74% of the tensile strength of the lower chord.



Fig. 4: Pin connection (lock) of the lower chord of the bridge ŽM 60.

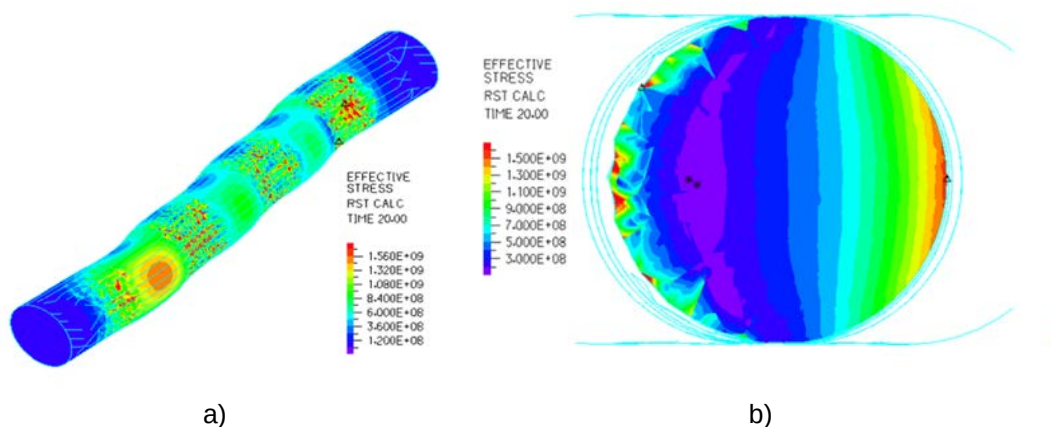


Fig. 5: Stresses on the pin surface a) and pin cross-section b) [20].

The rigidity of the joint is taken into account in the model by modelling two short 1D elements at the location of the upper and lower flanges of the lower chord cross-section. The 1D elements have a cross-section consisting of four sheets, which replaced the "ridges" of the locking joint. They are connected to the lower chord by rigid arms. The connection of two adjacent parts is modelled as hinged.

The self-weight of all structural members [23], the wind load [24] on the structure and on the vehicles or train and the traffic load are considered in the models [25]. Load schemes representing the traffic load on the bridge are given in the relevant sections of this article.

3.2 Stability of upper chord

As is obvious from Chapter 3.1 to 3.3, the limiting element of the structure is the upper chord for bridges with spans from 60.0 to 78.0 m. It is subjected to compression and with related buckling. All load-bearing capacities of the upper chord of the truss girder were determined under the assumption of using an original longitudinal bracing in the shape of a rhombic truss structure (Fig. 16). According to [15], this type of bracing is used for bridges that will perform their function for a longer time. For short-term deployment of the bridge, it is possible to use an upper longitudinal bracing consisting only of transverse frame members (Fig. 9). However, it is quite common for the bridge not to use the prescribed shapes of stiffeners in practice. This is most often the case with temporary bridges used on construction sites.

A study was prepared that considers the influence of different types of bracing of the upper chords of ŽM 60 on the stability and buckling resistance of these members for this reason. The out-of-the-plane truss buckling lengths of the upper chord were determined by means of a stability calculation for individual types of upper longitudinal bracing and different spans of bridges [26]. The out-of-the-plane buckling was a decisive stress type for the upper chord.

Six models were created for each range from 36.0 m to 90.0 m, differing only in the shape of the upper longitudinal bracing (Fig. 6). Considered types of bracing:

- Type A – Main girders with full longitudinal bracing (rhombic truss structure with transverse frame members).
- Type B - Main girders fully braced at both ends, intermediate segments braced by transverse frame members only.
- Type C - Main girders are fully braced at one end, and other segments are braced by transverse frame members only.
- Type D - Main girders braced by transverse frame members in each node of the upper chord.
- Type E - Main girders braced only by transverse frame members in every second node of the upper chord.
- Type F - Main girders without upper longitudinal bracing.

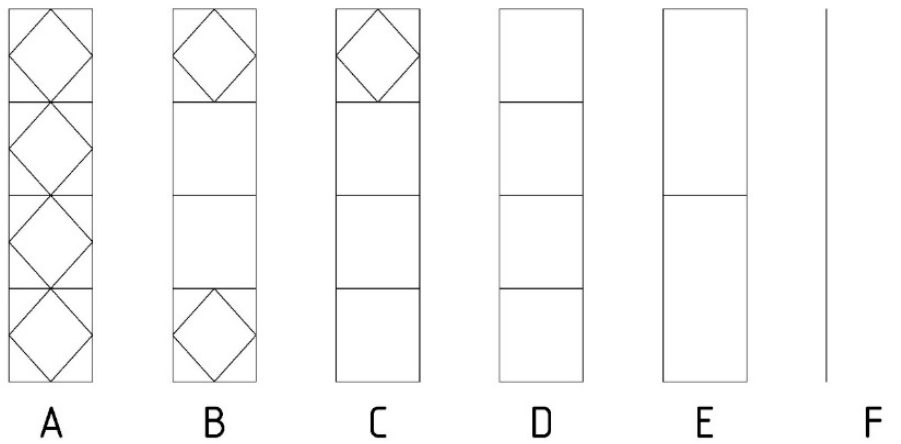


Fig. 6: Upper chord longitudinal bracing arrangement.

A critical factor α_{cr} was determined by the stability analysis for the shape of the out-of-the-plane deflection of the upper chord for all models. Subsequently, the critical Euler force N_{cr} and the buckling length L_{cr} for this buckling shape were calculated [27], [28].

The critical Euler force can be determined from the formulas:

$$N_{cr} = \frac{\pi^2 EI}{L_{cr}^2}, \quad (1)$$

$$N_{cr} = \alpha_{cr} N_{Ed}, \quad (2)$$

where:

N_{cr} – critical Euler force,

E – modulus of elasticity,

I – quadratic moment of the cross-section,

L_{cr} – critical buckling length,

α_{cr} – critical factor,

N_{Ed} – design value of axial force.

Then, it is possible to calculate the critical buckling length by combining both formulas as:

$$L_{cr} = \sqrt{\frac{\pi^2 EI}{\alpha_{cr} N_{Ed}}} \quad (3)$$

The values of the buckling lengths out-of-the-plane of main girders for the superstructure's individual spans and the different bracing types are given in Table 1.

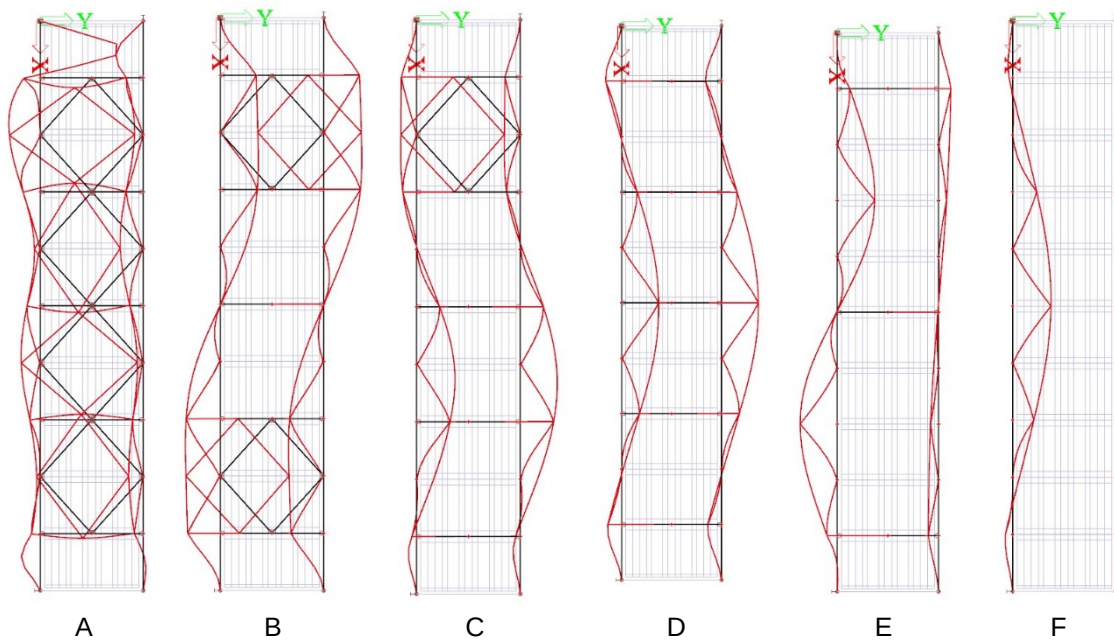


Fig. 7: The shape of the deflection of the upper chord out of the plane for the bridge with different upper longitudinal bracing.

Table 1: The buckling length $L_{cr,z}$ for out-of-the-plane buckling.

Span	Type of upper longitudinal bracing					
L [m]	A	B	C	D	E	F
30	5,693	7,681	8,159	8,891	10,809	13,491
36	5,696	7,995	8,309	8,950	10,796	13,598
42	5,751	8,302	8,650	8,999	10,771	13,954
48	5,888	8,713	8,911	9,053	10,761	14,522
54	5,711	9,095	9,113	9,116	10,752	14,898
60	5,546	9,224	9,201	9,210	10,966	15,188
66	5,621	9,354	9,320	9,354	10,908	15,123
72	5,559	9,506	9,457	9,403	11,139	15,088
78	5,498	9,644	9,609	9,579	11,205	15,220
84	5,456	9,757	9,815	9,653	11,222	15,447
90	5,533	9,793	10,040	9,666	11,254	16,029

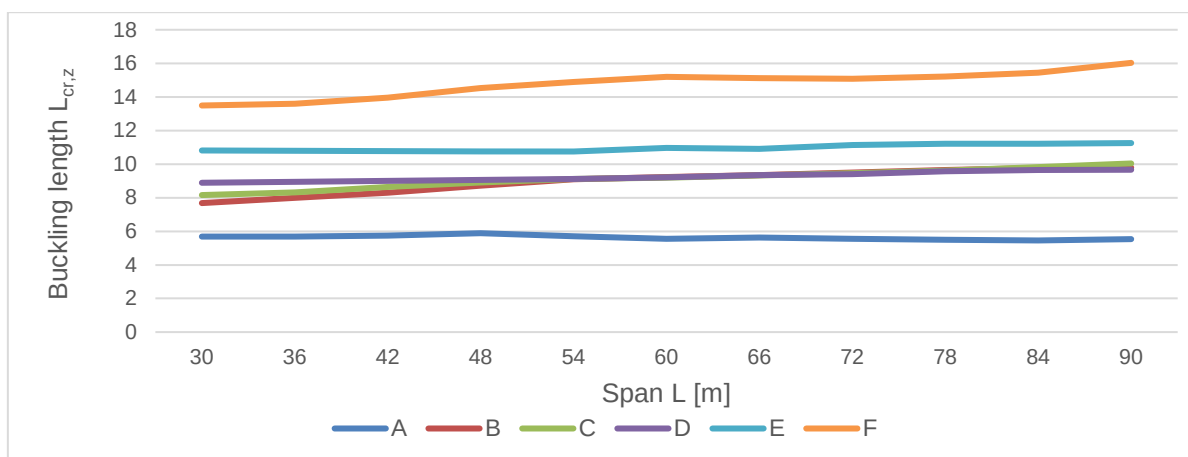


Fig. 8: The buckling length $L_{cr,z}$ for out-of-the-plane buckling.

From Table 1 and Fig. 8, it is obvious that in the case of bracing systems, the superstructure's span does not significantly affect the buckling length. With a fully braced system (type A), the out-of-the-plane buckling length is approximately equal to the distance of the nodes on the truss girders (6.0 m). For types B, C, and D, where the upper bracing consists mainly of transverse frame members and possibly fully braced segments at both or one end of the bridge, the buckling length is from 7.6 to 10.0 m. By placing the frame transverse members only in every second node of the upper chord (type E), the buckling length of the chord is increased up to 10.0 m, which is almost double compared to a fully braced structure. A significant increase in the buckling length at the bridge without the upper longitudinal bracing (type F) was observed. Especially in the case of bridges with a longer span, there is a significant decrease in the buckling resistance of the upper chord and a reduction in the load-bearing capacity of the superstructure.

3.3 Verification of the numerical model

The numerical model described in Chapter 3.1 was verified by the experimental investigation of the actual structure in situ [17], [29]. Several load tests were performed on real superstructures with different spans for this purpose. For illustration, the results of one of them are presented below. During the proof load test of the ŽM 60 temporary bridge with a span of 30.0 m (Fig. 9), the deflections of both main girders in the middle of the span from the test load were measured.



Fig. 9: Load test of ŽM 60 bridge.

Two four-axle vehicles with a weight of 32 tonnes were used as a test load (Fig. 10). The vehicles were situated with the rear axles close to each other and as close to the middle of the bridge span as possible.

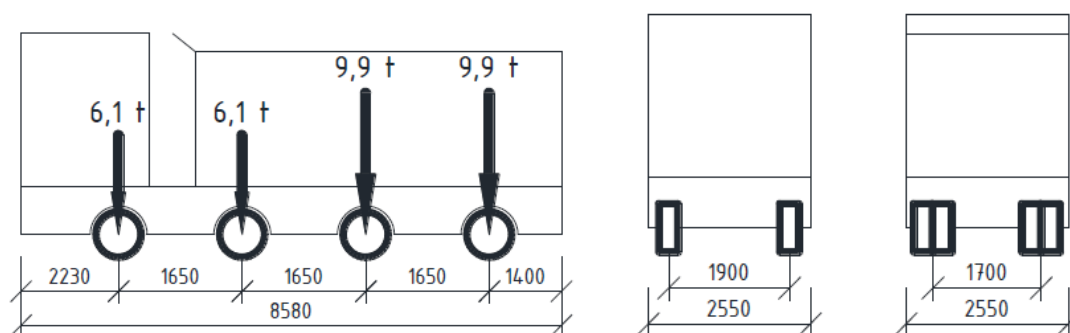


Fig. 10: Vehicle load scheme.

The main girders' deflection from the test load at the lower chord's location in the middle of the span was calculated. The deflection value reached 4.85 mm (Fig. 11). The following values of elastic deflection were determined by measurements of the real structure:

- 4.82 mm on the left girder (99% agreement with the model) and
- 4.23 mm on the left girder (87% agreement with the model).

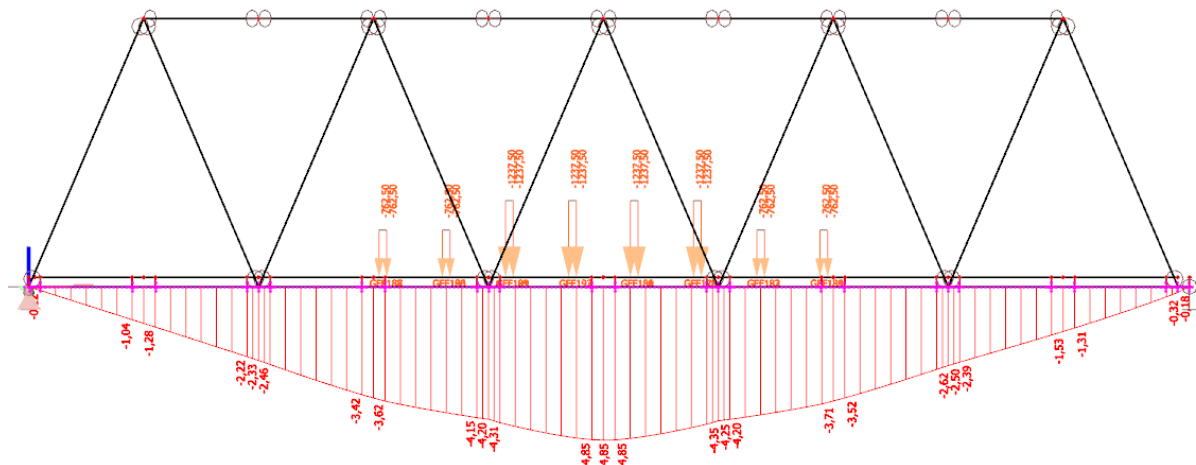


Fig. 11: Deflection of the lower chord of the main girder due to the test load.

The small deviation in the placement of the test vehicles on the bridge causes the different deflections of the right and left girder. In addition to this proof load test, measurements were also performed on temporary bridges with spans of 48.0 and 60.0 m. Similar results of measured and calculated deformations were obtained in these tests as well. It could be confirmed that there is sufficient agreement between the numerical model of the structure and its real behaviour based on these measurements.

4 Civil sector applications

In road transport, the ŽM 60 temporary bridge is used in emergency situations of permanent bridges as a temporary solution to overcome obstacles. The second most common application is the use of a temporary bridge as construction site equipment in the development of transport infrastructure. In addition, the temporary bridge ŽM 60 can also be used as a railway bridge.

4.1 Road bridge

There are several emergencies on road bridges, mainly prestressed reinforced concrete superstructures built in Slovakia in the 1950s. Due to neglected maintenance, serious defects arise in structures, which can result in the collapse of the structure. A temporary bridge is usually needed to provide an alternative detour route [30].

The failure of the bridge over the Studený potok watercourse in the village of Podbiel occurred in September 2015 (Fig. 12). The bridge is part of the main route to Poland. The collapse of traffic occurred in the region after the closure of the damaged bridge and the creation of an alternative 50 km detour route. The construction of a temporary bridge with sufficient capacity to carry traffic over the watercourse was a quick and affordable solution. ŽM 60 temporary bridge with a span of 60.0 m was used.



Fig. 12: The damaged bridge during demolition and the temporary bridge ŽM 60 [30].

According to regulation [31], normal, exclusive and exceptional load values must be determined before commissioning. The maximum value of the axle force that is permitted on the bridge is also determined. The load-carrying capacities determined according to this document thus include all standard vehicles whose total weight does not exceed the specified value. All 11 models described in Chapter 3.1 were analysed in this study focused on the investigation of load carrying capacity of ŽM 60 for standard road traffic. The load-carrying capacities of the individual members of the superstructure were determined using the appropriate load schemes. For this study, the following values were determined:

- Normal load-carrying capacity V_n was determined as the maximum permissible weight of one vehicle at normal load (without limiting the number and position of vehicles on the bridge). The LM1 model, according to [25] was used as the loading model,
- the exclusive load-carrying capacity V_r was determined as the maximum permissible weight of a single vehicle on the bridge. The vehicle model for exclusive load-carrying capacity corresponds to special vehicle class 900/150 according to [25] Art. A.2 (1).

The dynamic and horizontal effects of traffic loads were considered by the regulation [31] and the standard [25].

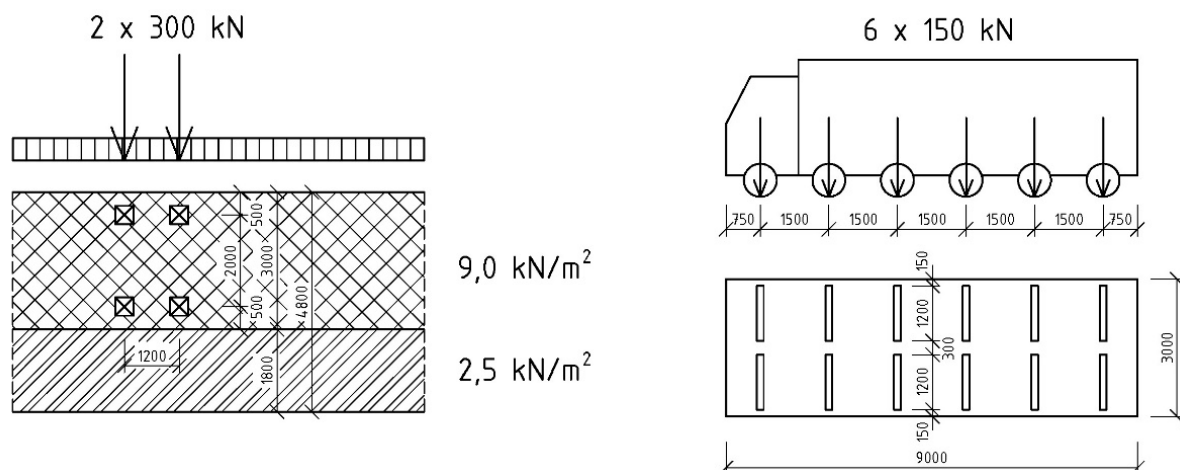


Fig. 13: Load models for normal and exclusive load-carrying capacity.

For individual load-bearing parts of the structure, the values of load-carrying capacity in tonnes are given in Table 2. Based on the graph in Fig. 14, it is possible to get a general overview of the possibilities of deploying the temporary bridge ŽM 60 for standard road traffic.

Table 2: Normal load-carrying capacity of temporary bridge ŽM60.

	Normal load-carrying capacity [tonnes]										
Bridge span [m]	30	36	42	48	54	60	66	72	78	84	90
Bridge deck	96.6	96.6	96.6	96.6	96.6	96.6	96.6	96.6	96.6	96.6	96.6
Diagonals	48.8	44.7	41.2	38.3	36.8	34.5	32.6	30.6	28.8	27.1	25.0
Upper chord	85.4	75.2	66.6	58.0	47.7	40.9	32.0	24.4	18.0	12.3	7.7
Lower chord	105.2	93.8	79.8	71.1	59.5	52.0	44.0	37.1	31.5	25.9	21.4
Joint	183.2	141.2	99.9	74.6	57.5	47.0	34.0	24.8	18.2	11.7	7.3

Based on the performed analysis, it is possible to state that the ŽM 60 bridge can be deployed up to a span of 66.0 m without limiting the weight of the traffic. It is already necessary to consider lower load-carrying capacity values for longer spans. The critical members are the upper chords of the main girders and the locking joints of the lower chords.

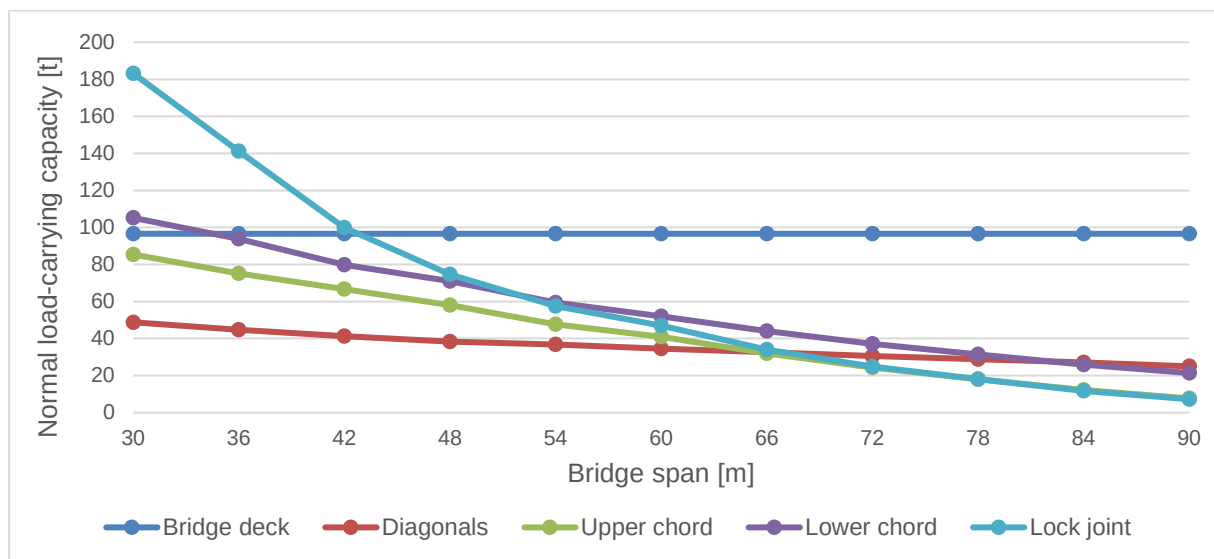


Fig. 14: Relation between normal load-carrying capacity and the span of the bridge.

Table 3: Exclusive load-carrying capacity of temporary bridge ŽM60.

Bridge span [m]	Exclusive load-carrying capacity [tonnes]										
	30	36	42	48	54	60	66	72	78	84	90
Bridge deck	218.0	218.0	218.0	218.0	218.0	218.0	218.0	218.0	218.0	218.0	218.0
Diagonals	131.1	129.8	126.1	121.1	114.4	111.1	107.9	104.3	101.7	98.1	92.7
Upper chord	205.1	192.3	174.5	156.8	143.9	133.4	117.1	95.1	74.6	54.3	35.3
Lower chord	316.4	292.2	248.6	218.9	200.7	184.5	160.2	140.6	125.8	108.0	93.2
Joint	453.1	399.9	321.7	238.4	188.3	162.2	124.5	96.0	75.5	51.3	33.3

The required value at the level of 48.0 t is exceeded at a bridge span of 90.0 m (Table 3). The resistance of the upper chord or the resistance of the joint on the lower chord is decisive for longer spans (Fig. 15).

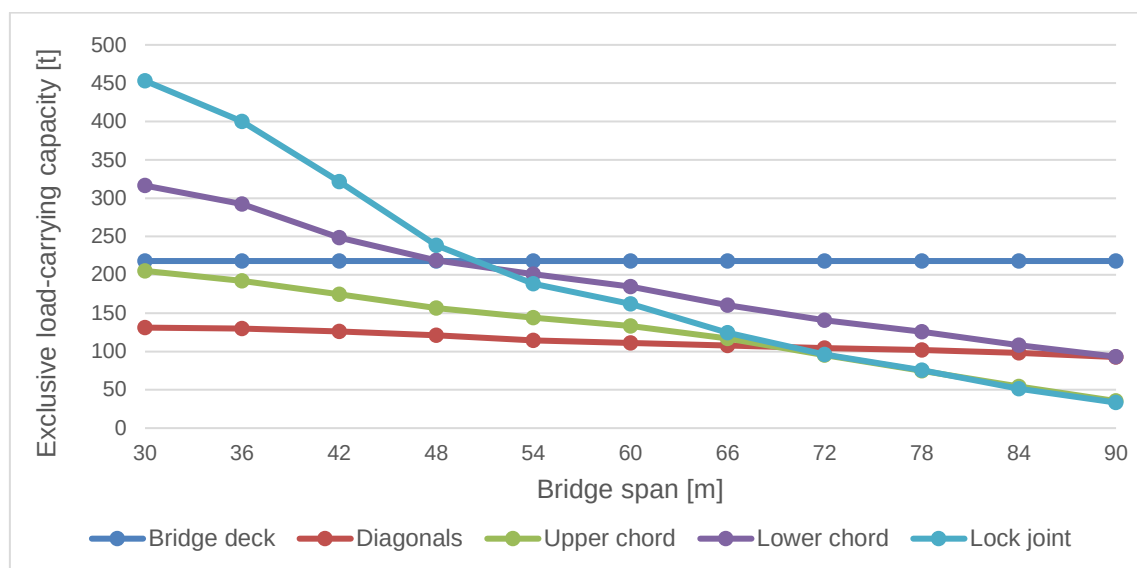


Fig. 15: Relation between exclusive load-carrying capacity and the span of the bridge.

4.2 Construction site bridge

Several new transport routes are being built in Slovakia, and existing sections are being modernised as part of the development of transport infrastructure. During construction, bridging over natural or artificial obstacles with a temporary bridge is often necessary. The construction of ŽM 60 appears to be advantageous. It allows the transport of heavy trucks with a high load capacity and the transport of oversized loads.

A typical example is the use of the ŽM 60 bridge in the construction of a railway bridge over the Nosický canal near Púchov (Fig. 16). The temporary bridge had a span of 90.0 m and was used mainly to transport large steel parts of the future railway bridge. The temporary bridge was in operation for four years.



Fig. 16: Temporary bridge ŽM 60 as part of the construction site.

Transport on the construction site has its own specifics. The transport fleet consists mainly of heavy trucks with significant axle forces. Two load models were considered as site traffic loads for the study:

- A four-axle vehicle model with a weight of 48.0 t (Fig. 17) and
- a group of four-axle vehicles with a 10.0 m gap between the bumpers of the vehicles.

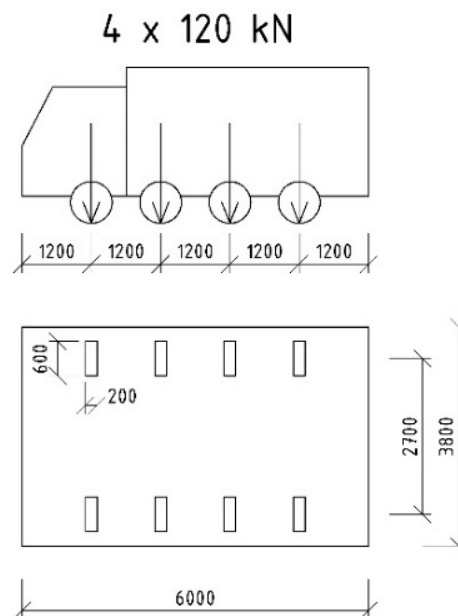


Fig. 17: Four-axle vehicle load model.

The FEM models described in Chapter 3.1 were used. Horizontal forces were determined according to [8]. The dynamic effects were taken into account by the dynamic factor, according to [31],

depending on the span of the structure. The procedure for determining the load-carrying capacity of such transport was the same as for road transport. The required load-carrying capacity of 40.0 - 48.0 t is mostly achieved in constructions with a span of up to 84.0 m (Table 4).

Table 4: Four-axle vehicle load-carrying capacity of temporary bridge ŽM 60.

	Four-axle load-carrying capacity [tonnes]										
Bridge span [m]	30	36	42	48	54	60	66	72	78	84	90
Bridge deck	186.2	186.2	186.2	186.2	186.2	186.2	186.2	186.2	186.2	186.2	186.2
Diagonals	120.8	114.8	112.7	107.5	105.4	102.3	99.2	95.9	94.0	90.7	85.6
Upper chord	198.0	169.6	153.8	144.7	138.4	128.4	118.5	97.5	76.3	55.2	35.9
Lower chord	236.2	210.0	191.2	176.3	162.1	150.4	132.3	116.4	105.6	91.3	79.3
Joint	456.0	373.5	299.8	238.1	191.2	164.0	126.0	96.4	76.0	51.8	33.4

The decisive is the buckling resistance of the first pressed diagonal at smaller spans. For longer spans, the resistance of the lower chord locking joint is crucial. The load-carrying capacity's dependence on the bridge's span is clearly shown in the graph in Fig. 18.

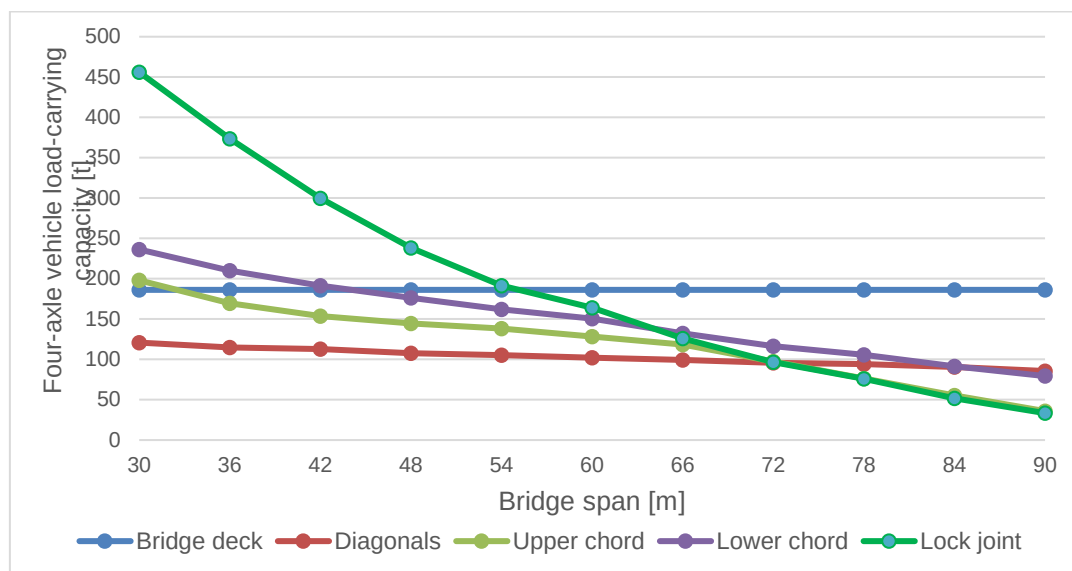


Fig. 18: Relation between four-axle vehicle load-carrying capacity and the span of the bridge.

Especially with longer spans of bridges, there is a risk that more vehicles will be on the bridge due to driver indiscipline. Therefore, the study for the case of a group of vehicles moving at a distance of 10.0 m in a row was conducted. The results are shown in Table 5.

Table 5: A group of four-axle vehicle load-carrying capacity of temporary bridge ŽM 60.

	Group of our-axe load-carrying capacity [tonnes]										
Bridge span [m]	30	36	42	48	54	60	66	72	78	84	90
Bridge deck	186.2	186.2	186.2	186.2	186.2	186.2	186.2	186.2	186.2	186.2	186.2
Diagonals	104.0	93.3	85.0	80.1	71.9	68.3	63.8	59.9	56.2	52.6	48.0
Upper chord	198.0	159.5	127.6	107.1	84.2	67.0	54.7	38.5	27.7	18.8	11.6
Lower chord	236.2	199.9	168.7	136.5	115.8	95.7	77.9	65.0	53.2	43.2	35.4
Joint	456.0	365.9	281.4	186.2	119.8	77.2	55.8	41.1	28.0	18.0	10.9

The load-carrying capacity decreases sharply, especially at spans with a risk of more vehicles on the bridge. Bridges with a span of up to 66.0 m are suitable for the required load capacity of 48.0 tonnes. At small spans, the buckling resistance of compressed diagonal is decisive again. At spans from 60.0 to 78.0 m, the buckling resistance of the upper chord of the main girder is decisive. The joint of the lower chord is critical for long spans above 84.0 m (Fig. 19).

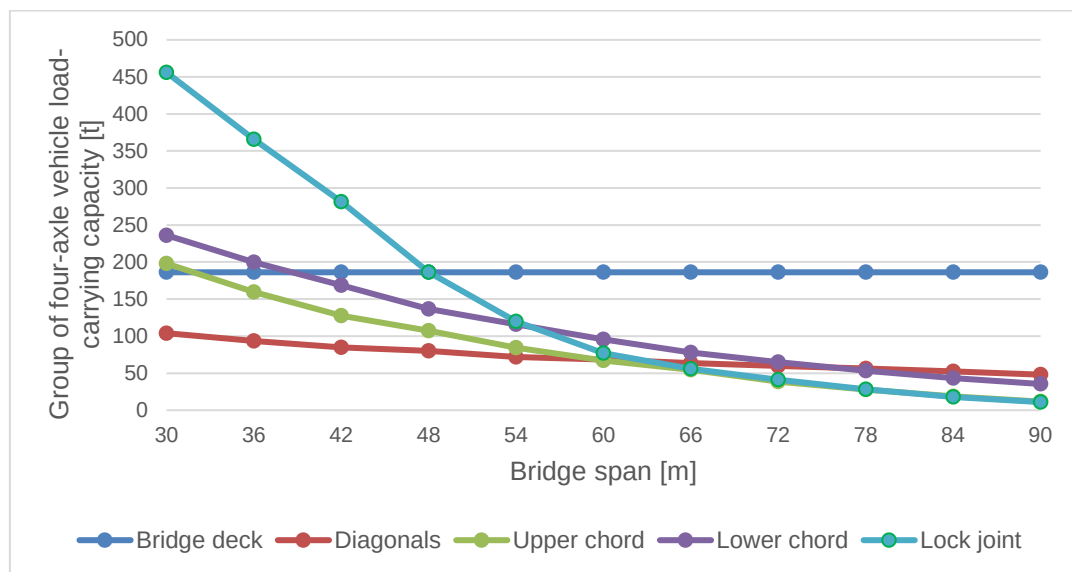


Fig. 19: Relation between a group of four-axle vehicle load-carrying capacity and the span of the bridge.

4.3 Railway bridge

As the introduction mentions, the temporary bridge ŽM 60 can also be used for rail transport. Load-carrying capacities were determined based on regulations for determining the load capacity of railway bridges [32], [33], [34]. The vertical traffic load is defined by load model 71 (Fig. 20), which also took into account the eccentricity due to the unevenly distributed load. Dynamic effects were taken into account by calculation using the dynamic factor. Horizontal effects were considered: nosing force, braking and traction forces. Although it is possible to create a track in the directional arc on the bridge, centrifugal forces were not considered.

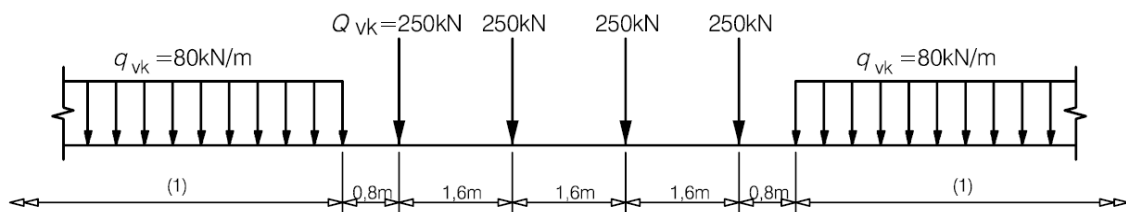


Fig. 20: Load Model 71.

All load-carrying capacities Z_{LM71} are less than 1.0 (Table 6). We would achieve higher load capacity values using some reductions, such as a reduction in the partial factor due to the age of the structure. Moreover, it is necessary to determine the passage of traffic load for all elements where the load capacity Z_{LM71} is less than 1.0.

Table 6: Rail traffic load-carrying capacity of temporary bridge ŽM 60.

	Rail traffic load-carrying capacity Z_{LM71}										
Bridge span [m]	30	36	42	48	54	60	66	72	78	84	90
Bridge deck	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98
Diagonals	1.09	0.99	0.94	0.87	0.81	0.76	0.71	0.66	0.62	0.58	0.55
Upper chord	1.73	1.45	1.21	0.98	0.75	0.65	0.43	0.32	0.23	0.15	0.12
Lower chord	2.20	1.89	1.59	1.31	1.04	0.88	0.70	0.65	0.46	0.37	0.31
Joint	2.56	2.05	1.58	1.13	0.76	0.61	0.42	0.31	0.22	0.14	0.11

It should be pointed out that in the case of railway transport, several elements of the structure are decisive due to the load-carrying capacity of the temporary bridge ŽM 60 (Table 6 and Fig. 21). For small spans up to 42.0 m, the limiting element is the bridge deck, for medium spans up to 48.0 m it is the compressed diagonal, and for longer spans, it is the upper chord and the joint of the lower chord.

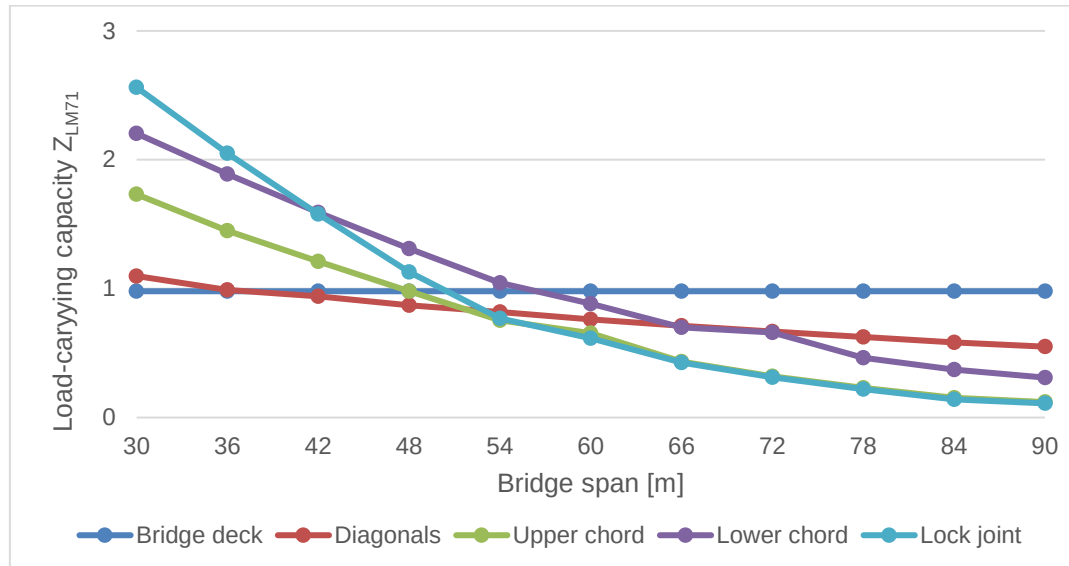


Fig. 21: Relation between a rail traffic load-carrying capacity and the span of the bridge.

5 Military bridge

The temporary bridge ŽM 60 also performs tasks falling within the scope of the army. It can be deployed in the rear positions of the army or during rescue operations. It is necessary to know what maximum military load the bridge can carry depending on its span, type of load, and accepted risk of using the structure.

Since March 2003, the Slovak Republic has been a part of the North Atlantic Treaty Organization. The traffic load of NATO military equipment is defined by the STANAG 2021 [35]. This standard defines hypothetical wheeled and tracked vehicles for the Military Load Classification (MLC). It divides the military load into individual classes, marked from MLC 4 to MLC 150. A tracked and wheeled vehicle is defined for each class (Fig. 22). Vehicles have defined dimensions, weight, the centre of gravity position, windward area, etc.

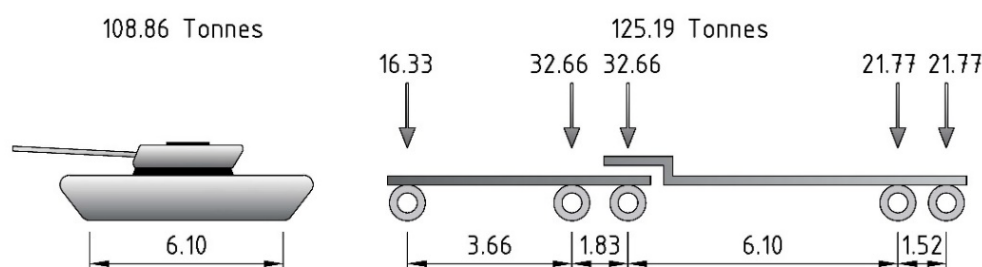


Fig. 22: Model of MLC 120 according to STANAG 2021 [35].

The STANAG 2021 [35] also provides regulations regarding different modes of the crossing of military vehicles over bridges:

- **Normal Crossing:** The normal crossing allows for unrestricted use of the bridge by military traffic and all vehicles or convoys operating at or below the maximum allowable MLC. The vehicles are in a convoy with a gap of 30.5 m between the vehicles that move over the bridge in any part of the road.
- **Caution Crossing:** The caution crossing allows for a higher allowable MLC by limiting the maximum speed to 5.0 km/h and restricting the use of braking, accelerating and switching gears. Vehicles must drive along the centreline and are only allowed to cross one at a time across each structurally independent span.

- **Risk Crossing:** Risk crossing allows for the transportation of higher MLC vehicles by adapting the same conditions as the caution crossing but additionally decreasing the minimum required safety. It greatly increases the probability of failure, and even if the bridge does not fail, permanent damage to the bridge may occur.

The different level of reliability of the bridge for individual types of crossings is given by the values of the partial factors (Table 7).

Table 7: Values of partial and dynamic factors.

	Traffic load by STN EN	Military load by STANAG 2021		
		Normal crossing	Caution crossing	Risk crossing
Permanent load	$\gamma_{F,G} = 1,25$	$\gamma_{F,G} = 1,25$	$\gamma_{F,G} = 1,25$	$\gamma_{F,G} = 1,25$
Traffic load	$\gamma_{F,Q} = 1,35$	$\gamma_{F,Q} = 1,10$	$\gamma_{F,Q} = 1,00$	$\gamma_{F,Q} = 1,00$
Wind	$\gamma_{F,W} = 1,45$	$\gamma_{F,W} = 1,45$	$\gamma_{F,W} = 1,45$	$\gamma_{F,W} = 1,45$
Material reliability	$\gamma_{M0} = 1,00$	$\gamma_{M0} = 1,00$	$\gamma_{M0} = 1,00$	$\gamma_{M0} = 1,00$
	$\gamma_{M1} = 1,10$	$\gamma_{M1} = 1,10$	$\gamma_{M1} = 1,10$	$\gamma_{M1} = 1,10$
	$\gamma_{M2} = 1,20$	$\gamma_{M2} = 1,20$	$\gamma_{M2} = 1,20$	$\gamma_{M2} = 1,20$
Dynamic factor	$\varphi \geq 1,00$	$\varphi = 1,10$	$\varphi = 1,00$	$\varphi = 1,00$

According to [25], [36], the horizontal load is considered:

- braking forces with an intensity equal to 60% of the vertical load in the longitudinal direction,
- transversal forces with an intensity equal to 10% of the vertical load in the transverse direction.

In addition, the infantry load and the weight of mud or snow on the bridge were modelled as a separate load case. A uniform surface load of 5.0 kN/m² was considered.

It is possible to find a hypothetical MLC vehicle, the effects of which the structure is still able to transfer under the given passage conditions. Assessments of load-bearing capacity of main elements of the structure were elaborated for individual spans and individual MLC classes. Table 8 clearly shows the possibilities of using the ŽM 60 bridge for military purposes. For smaller and medium spans, the bridge meets the conditions for the MLC 70. For higher classes of military transport (MLC 90 and MLC 100), the members fail due to the high values of vehicle axle forces and their horizontal effects. The critical element of the bridge is the bridge deck in this case. For classes, MLC 120 and MLC 150, a lane width of at least 5.0 m is required, which the ŽM 60 bridge does not meet.

Table 8: Passage of MLC vehicles over the temporary bridge ŽM 60.

[illegible]

	suitable for normal crossing
	suitable for caution crossing
	suitable for risk crossing
	not suitable

6 Conclusions

Based on the analyses, it can be stated that the temporary bridge ŽM 60 has a wide scope of application. Originally designed for rail and road transport, it can be used for military tasks or as part of a construction site. By applying the currently valid standards and regulations for loading and assessing structures, its behaviour was verified, and the conditions of its use in terms of structural resistance were determined.

For spans up to 60.0 m, the bridge is suitable mainly for regular traffic. For longer spans, transport is possible, but with restrictions. The type of bracing of the upper chord of the main girders must also be taken into account precisely. The buckling resistance of these elements can be a limiting factor in determining the load capacity of the bridge's superstructure. It should be noted that the structure's load capacity was determined under certain conditions in individual cases. Mainly, the perfect condition of the individual parts of which the construction consists was considered. It is necessary to verify the construction by global analysis at each use. The analysis must also take into account the various effects that could reduce the load-carrying capacity of the bridge superstructure. This is mainly the effect of corrosion of steel members and deformation of parts, which were caused by transport or previous deployment of the bridge, etc.

In the analysis of the ŽM 60 bridge, as with all temporary bridge structures, a fundamental problem occurs. This problem is the assessment of the fatigue resistance of the structure [37]. To verify it, we would need records of using the entire structure and its parts. However, this is not possible in practice, and therefore, this problem is resolved by frequent inspections and checking of details liable to fatigue.

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