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New Troja Bridge over Vltava River in Prague

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ABSTRACT

The structure of the new Troja Bridge crosses the Vltava River on northern part of city centre. The bridge has two spans – the main span is crossing the river and is 200,4m long and the side span is 40,4m long. The main span is crossed over by steel network arch, which is extremely flat (rise/span ratio is 1/10), and by the suspended concrete deck with tie. The side span is single span completely in situ cast prestressed concrete structure. Final structure behaviour and construction process of the bridge is very complex and difficult. It was necessary to use lots of computational models for simulation and prediction of structure behaviour. The results from mathematical simulation were continuously compared with measurement results and computational models were continuously updated. The construction process is now almost finished.

1. INTRODUCTION

In the year 2006 was announced architectural competition by the client – City of Prague. The winning project was submitted by J. Petrák and L. Šašek (Mott MacDonnald, Prague) and R. Koucký and L. Kábrt (Koucký arch. office, Prague). The Novák&Partner Company proposed the incremental launching construction process, temporary structures and under the terms of supervision made computational analysis of the structure with respect to all construction stages. The structure of the new Troja Bridge connects central part of the city with the city ring road. The bridge has two spans – the main span is crossing the river and is 200,4m long and the side span is 40,4m long. The bridge should be opened in 2013.

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Fig.1 Architectural concept of the bridge

The main span is crossed over by steel network arch, which is extremely flat (rise/span ratio is 1/10), and by the suspended concrete deck with tie. The bridge bears two tram tracks, four road lanes and two pedestrian lanes. The steel arch has a multiple box section at the midspan. The section splits into two legs close to the supports. The arch footings are fixed to the concrete deck and to the last massive in situ cast transversal beam. Because of extreme load the footings are filled with self compacting concrete of the strength class C80/90. The main span concrete deck is composed of a thin in situ cast slab (C50/60), with typical thickness 280mm. The deck is stiffened by precast prestressed transversal beams (C70/85), which are only 500mm wide and almost 30m long with the weight 50t and they are suspended by network hangers on tie. In longitudinal direction the deck is stiffened only by two ties of arch with composite cross section. The inclined hangers are in diameter range 76-105mm. Each transversal precast beam is prestressed by two cables with nine strands. The concrete bridge deck is heavily prestressed (VSL system). The transversal prestressing tendons are composed of four strands 15,7mm in flat ducts. The longitudinal prestressing is rather complex. Six cables with 37 strands are located in each composite tie. The slab is prestressed by number of cables with 7 to 22 strands. The pedestrian stripes are located on the steel cantilevers, which will be attached to the edge stiffening concrete beam of the bridge deck. The side span is the Inundation Bridge. It is single span completely in situ cast structure made of the prestressed concrete. The side span bridge is shaped similarly to main span concrete deck. Main span and a side span are separate structures supported on individual bearings and mutually connected on the intermediate pier only via expansion joint.

2. FEASIBILITY STUDY

Because of the great public interest and great importance of the Troja Bridge structure there has been set great demands on confirmation its feasibility. The feasibility has been demonstrated on a mathematical model and on the model testing of some structural details. For obtaining forces to the foundation was created 3D model of the structure. The model was set as TDA (Time Dependent Analysis). TDA in this model was used to just for determine superload respecting each phase of construction.

To ensure proper strength parameters of concrete components in the model have been considered aging effects of concrete. For calculating effects of creep and shrinkage has been created another more simplified 2D beam TDA model.

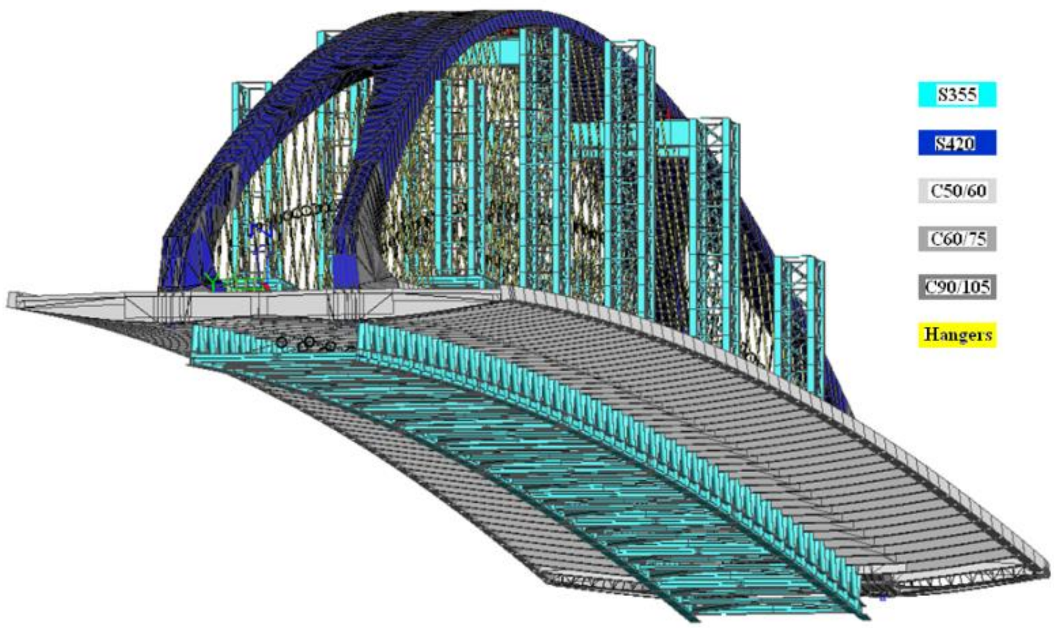

Fig.2 Visualization of mathematical model for feasibility study and used materials

There was also solved vertical deformation of the arc structure. In the original proposal show a significant bending effects, therefore was centerline aligned. The centerline of the arch is designed to be bendless, in the arch take effect mainly the normal forces and thus the vertical deformation is being reduced. On the models where have been analyzed values of the tie extension was also established deflection curve, which is dependent on the axial stiffness of the tie (with respect to the proposed "bendless" shape). On the Fig. 3 and Fig. 4 is showed comparison of the vertical deformation originally proposed version and realized aligned centerline.

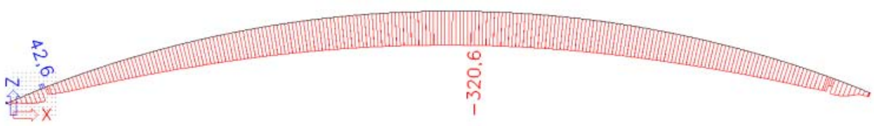

Fig.3 Vertical deformation of the original proposal (comparative model)

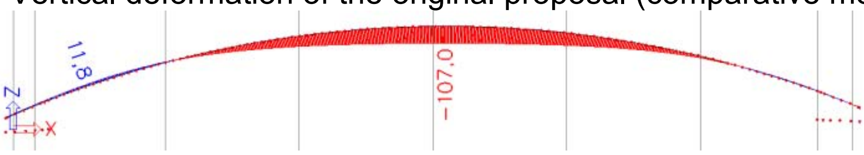

Fig.4 Vertical deformation of realized aligned centerline (comparative model)

3. CONSTRUCTION STAGE ANALYSIS

Proposal of the temporary structures and incremental launching analysis

The bridge construction method is based on temporary steel truss structure in the static scheme continuous beam situated under the entire proposed structure. This truss will create sufficient rigidity of the supporting structure for the further stages of construction. This temporary truss was launched from the Holešovice shore through set of seven temporary support - outer two are located on the shore (P01 and P07 in the area of the final abutment O1, respectively columns P2) and five (P02 to P06) is situated in the river.

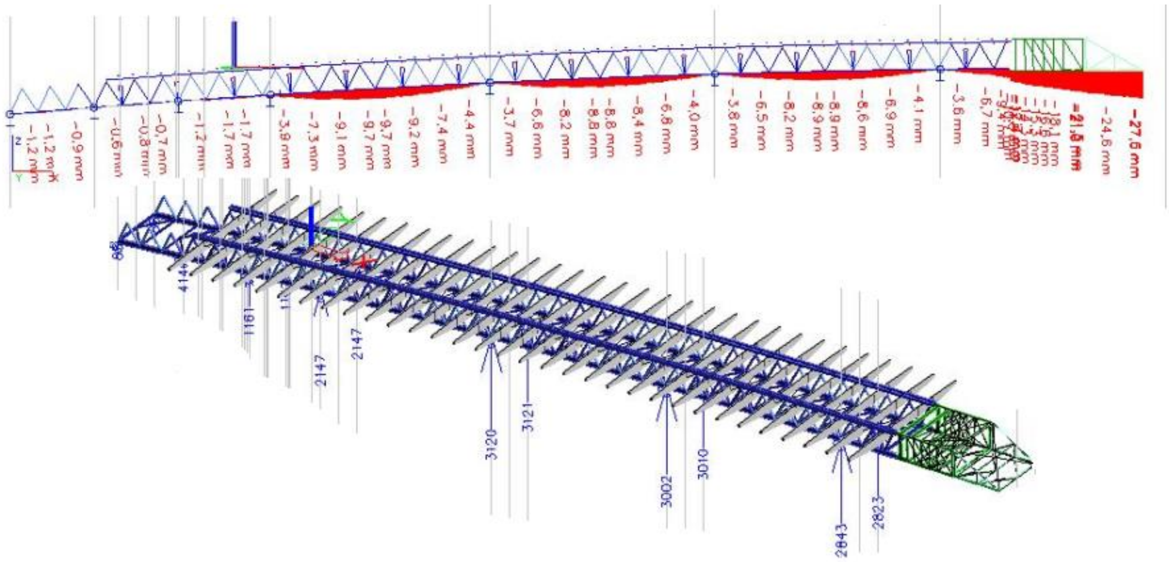

Fig.5 Visualization of the matematical model used for assesment construction stage of incremental launching

For proposal of temporary structures was considered 34 stages of construction in feasibility study, for assessment was made 87 separate model for each phase from global 3D model. When the load was determined then could be created 3D shell elements model temporary supports and 3D planar elements models for the assessment of structural details.

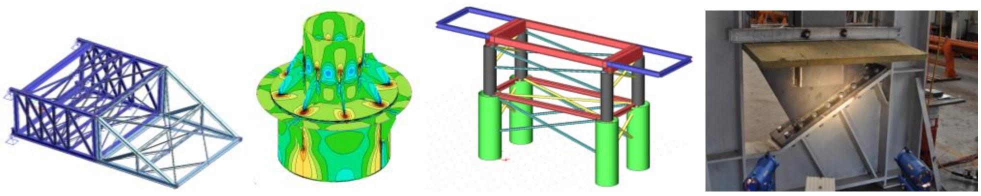

Fig.6 Assessment of the structural details

Analysis of the further stages of construction

After the end of the incremental launching temporary truss was performed mounting the arc rudiment and formwork of monolithic terminal transversal beams. This was followed by sequential casting of composite deck and tie of the bridge according to defined casting shapes. Rudiments of the arch were also filled with SCC. During casting of the deck was applied transversal prestressing. After completion of superstructure was a partial activation of longitudinal prestressing system (about 1/3 of the total prestressing). Then was launched assembly of steel construction of the arc. The individual components were placed with help of the loading tracks and lifting via temporary truss support which were placed on the deck. All structural components have been assessed on this specific load case. For assessment were created simplified 2D TDA beam model with consideration of a procedure of construction thus loading history of concrete elements to ensure correct stiffness of the construction in each phase (e.g. the casting slabs, prestressing...). The result were verified on a 3D spatial model, but due to the limited options TDA module on planar elements, was considered in this model only a function of aging concrete.

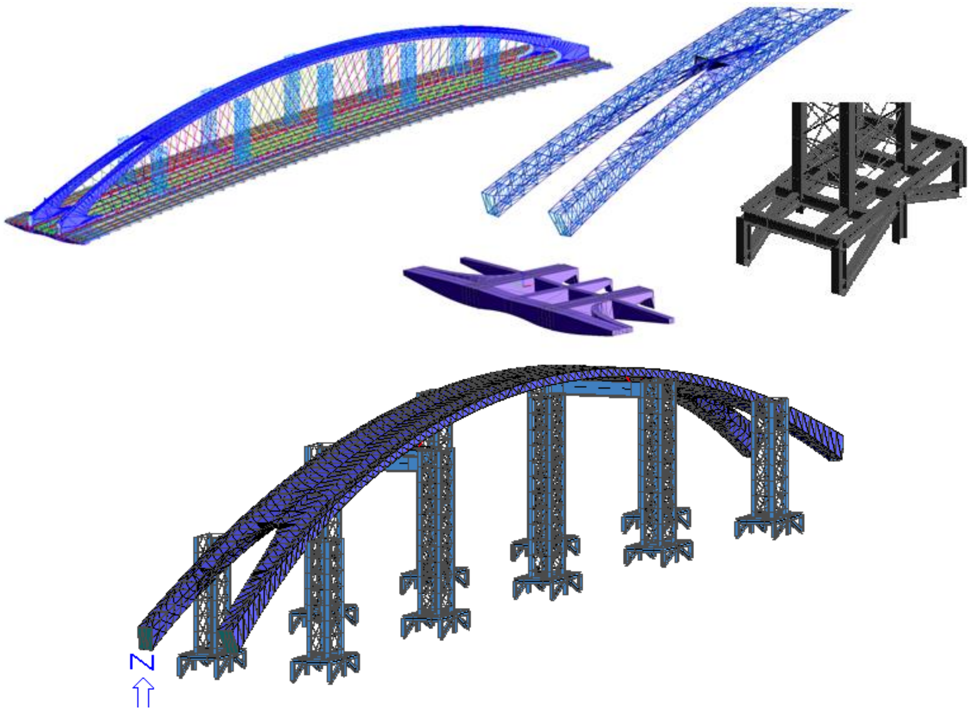

Fig.7 Visualization of the mathematical model of structure and its parts

In this section were mainly assessed phases of construction taking place after the incremental launching. It was assessed mainly casting of bridge deck, prestressing, handling the mounting parts of the steel arc, temporary mounting towers and the actual effect of assembling the steel arc on the others parts of the structure.

Analysis of the hangers tensioning

This phase of construction is fundamental with regard to verifying correctness of the structural system behavior that requires adequate reserve of internal tensile forces in each hanger. A sufficient reserve of tension is necessary to prevent deflection of the hanger dead load. The cable elements occurs to reduce their stiffness due to their sagging - reduction of substitute modulus of elasticity. The applied normal force is depleted changing the geometry of the rope element, to the point where the rope is taut. Since that time we can assume that the element behaves linearly and the rope operates with full rigidity. This behavior is generally referred to as tension stiffening. For single ropes, characterized by their proposed geometry, was determined value required tensile force to guarantee their almost linear behavior.

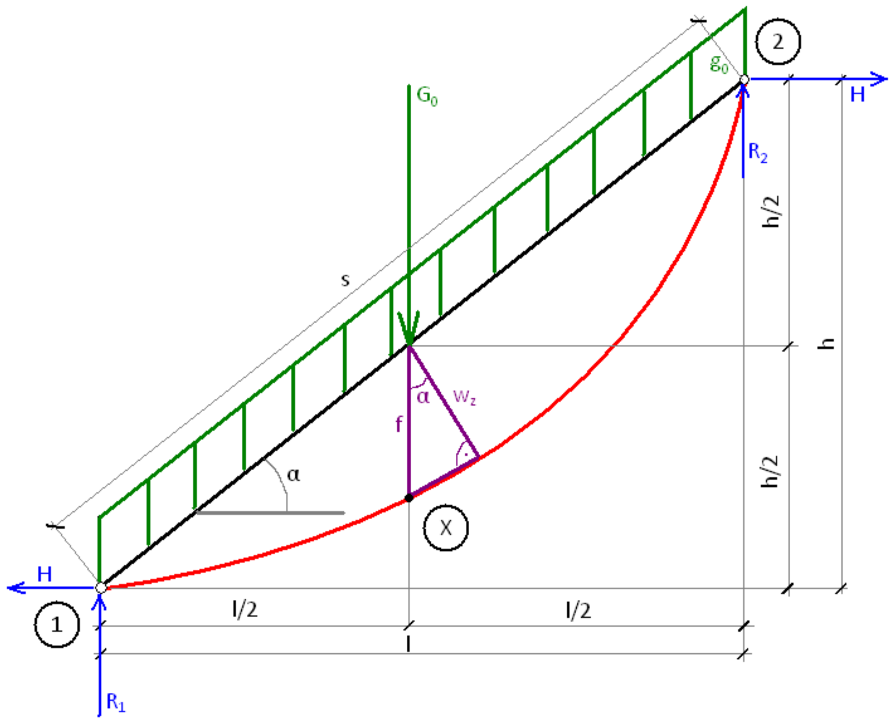

Fig.8 Scheme of calculation non-linear stiffness of the hangers

For the forecast the behavior of the structure was formed 44 phases tensioning the cables. Searching internal forces is an iterative process in each stage is added four ropes to the stiffness matrix of the entire system. The load is the considered as the dead load of the hangers and by shortening of the hangers (assigned as strain). For describing the interaction of hanging ropes (one rope tension effect on the rest of the system) was prepared "influence matrix". Due to the influence matrix was proposed rectification process.

Temporary truss disassembling study

For the proposal of lifting is fundamental size of the expected deformation during the process and the evolution of vertical reactions on of temporary supports in the river and definitive bearings. Evolution of the reactions and their redistribution is a key indicator in the process of the static system change. Controlled deformation leads to unloading of the temporary supports in the river and moving the stress over activated suspensions to the arch and through the arc to the final bearings. The structure progressively takes the final form of the static effect. Due to the basic static principle of the structure - arch with rod - lifting leads to the initiation of tensile force in the bridge rod - and thereby stretching the structure. This extension will be possible to control by means of integrated tensionmeters and measuring actual stretching of deck. It is designed a total of 34 steps lifting of the bridge - the progressive deactivation of temporary supports in the river with their forced deformation – declination. The proposal of stepping is based on the premise to not encumber the neighboring supporting structures unequal declination of more than 20 mm and the assumption that the maximum mutual difference between in the left and right bearings of the bridge on one support will not be more than 5 mm.

Proposal for a number and distribution of hydraulic presses are based on load - size of the vertical reactions per individual support and capacity of the presses. Estimated reactions per calculation are confronted with real values of reactions that are detected during mounting of hydraulic presses instead of temporary bearings. The first was the support P05. According to the measurement of the hydraulic activation process dated January 29, 2013 is the reaction 7147 kN. According to the results of computer analysis (with respect to the first third of the longitudinal tension) is the average response to the left and right bearings 7261 kN, which is in contrast to measurements +1.6%.

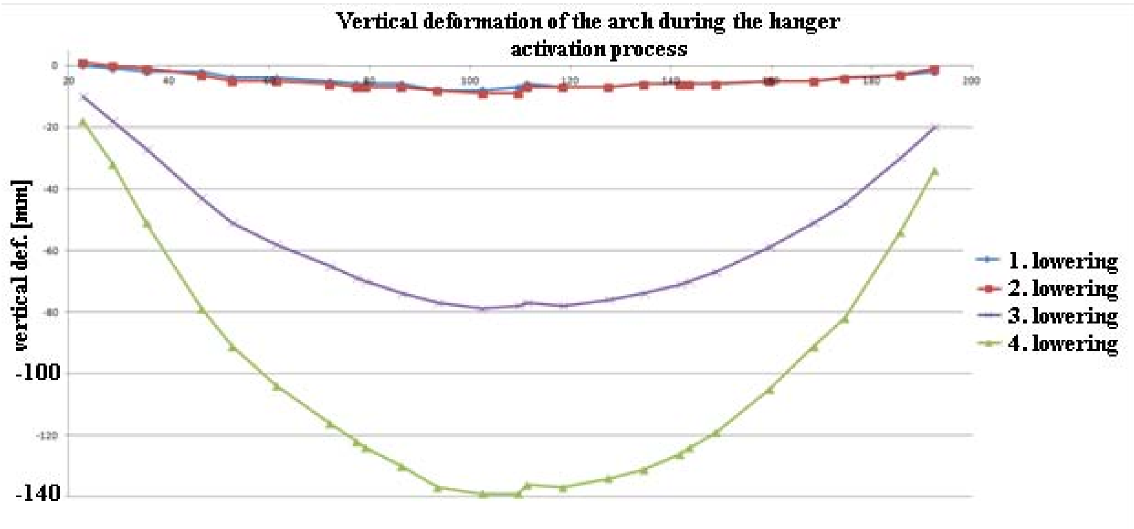

Fig.9 Vertical deformation of the arch during the hanger activation process

The two specific details exposed during activation lifting device were assessed. Solved was contact of presses with joints of the temporary truss lower waist and contact of presses with welded beam of temporary supports in the river. For each detail has been created a separate shell model that respects geometry according to the technical documentation. Load acting on the presses is taken from the phased model according to the relevant stage. In the value of the load is considered reserve 30% due to non-uniformity forces in presses, evaluated are stress values of the characteristic load combination.

4. GLOBAL STATIC AND DYNAMIC ANALYSIS OF THE STRUCTURE

Simplest 2D beam model, where all the construction parts were simulated by the beam elements, was primary used to TDA module analysis of the construction process, taking into account the influence of creep and shrinkage. The others models were rather more complex. In the main 3D model were used mainly planar 2D elements, only for hangers and temporary truss were used beam elements. In this model has been defined 11 569 planar elements, 4 719 beam elements with 107 cross sections, 19 089 knots, 7 materials and 107 load cases. This model was used for the global static, dynamic, non-linear (geometric and material non-linearity) and non-linear stability analysis. The model served also as the basis for the detailed design aerodynamic stability of the structure. In calculations of geometric nonlinearity was considered a solution according to the 2nd order theory. Nonlinear solution of the sagging suspension elements with axial tensile force was made with respecting the tension stiffening theory. All the results were compared with simplified calculations on models, where is known exact analytical solution. This model was also used to propose rectification process of the hangers. Hangers of the bridge were modeled as nonlinear elements - with sagging beam elements able to transmit only tensile stress. For single ropes have been calculated their Influence surfaces for determining the effects of moving load and it was estimated their force reserve.

Fig.10 View of the structure with hangers

Main 3D mathematical model of the bridge structure has been used for the analysis of the dynamic effects of moving loads. The focus of solving the dynamic behavior of the system was mainly on dynamic response of bridge structure. Increased attention has been given to the possible loss of aerodynamic stability. Simplified calculations based on empirical relationships were made, but it also held a detailed analysis measurement of the sectional model and subsequent numerical calculations, which confirm that the loss of the aero dynamical stability occurs when speed of the wind is much higher than the norm rate for the area. Calculation of the stability of the structure was carried out according to the conventional manner Eulerian approach to stability caused by the bifurcation of equilibrium - sought the multiplier of the load at which stability is lost.

5. CONCLUSION

For the assessment feasibility and construction phases of the bridge were developed spatial models with beam and planar elements that correspond to the proposed geometry and material characteristics of the bridge. Structural measuring at specific stages shows a very small difference between the mathematical models of the structure and its actual response. The calculations under the project supervision demonstrated the feasibility of the design and the adequate reserve of reliability.

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