Potential modelling and verification of bridge superstructures behavior

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Abstract. During design and analysis process, many simplified technical approaches are routinely used in the application of theories to practice. With the use of modern structural analysis computer programs, the reliable design alternative, providing the most probable response of a bridge structure due to a range of designed loads can be identified. Field measurements should be performed to determine the actual load effect and to verify the applied analytical models. Modelling principles and suggest some guidelines as well as consideration in finite element based structural analysis are illustrated in the paper on selected bridge structures. Measurements for monitoring structural response, collected data are shown using tests carried out on composite bridge superstructures.

1. Introduction
Design of bridges is a complex task. Initially, the selection of modelling methodology, nowadays most often based on finite elements, is essential and followed by a description of the structural geometry, definition of the material and section properties of the components making up the structure, next by description of load situations acting on the structure. Conditions of the piers or abutments at the support points must be examined and implemented into the structural analysis model properly. With the increased power of computers, coupled with higher demand placed on the accuracy and efficiency of structural designs, more-detailed understanding of the basic principles and assumptions associated with the use of modern computer programs for structural analysis is required.

Obviously, when designing, there is not usually enough time to verify all details. In many cases, design of certain parts of the bridge structure is executed on the basis of experience. As a result numerous simplifying assumptions are used in a multitude of analytical methods available to structural analysts. Therefore loading tests are habitually necessary, especially in the case of bridge superstructures of unusual conception. The main purpose of testing is to verify calculation model and assumptions of designer as well as suitability of transformation model. To satisfy all required code criteria, still more sophisticated analyses are needed. In the paper, three rather uncommon bridge superstructures are presented. Firstly a composite overpass, next a superstructure consisting of steel grid with concrete deck and finally quite large double beams highway bridge structural modelling as well as experimental assessments are given.
2. Steel-concrete highway overpass

2.1. Structural concept
Composite bridge, whose concrete deck is supported by two steel girders, which carries road to slim piers, was especially preferable to common prestressed concrete bridge [1]. After consideration of the technical aspects, it was decided that the bridge structural system would consist of a total of four spans with lengths 21.0+31.189+23.811+21.0 m. The roadway has a width of 7.2 m between barriers and the total deck width is 8.7 meters. Thus, this composite superstructure defines the two-lane overcrossing over its whole 103.0 m length. The steel built-up girders of I-section, axially 4.0 m spaced were selected for the bridge having the variable slender web depth from 1.30 to 1.80 m with parabolic haunches over piers (Figure 1). Transverse stiffeners welded to inner sides of the web were placed for meeting slenderness requirements and preventing local buckling. The inconstant area of flanges was used to save material where the bending moment would be smaller or larger in a span. Especially, the top flanges that act with the concrete slab are of the constant 350 mm width and proportioned by varying thickness from 20 to 50 mm. For increasing the flexural strength of cross-sections, the bottom flanges are 650 mm width, and vary in the thickness from 30 to 40 mm. Low-alloy structural carbon steel S355 grade has been used for steel bridge structural parts. Bracings consisting of rectangular hollow diagonals and horizontal channels and acting as a truss provide lateral stability of the girder bridge and distribute vertical loads. Reinforced concrete with 28-days compressive strength 35 N.mm\(^2\) was used in 330 mm thick slab with haunches near girders. Shear stud connectors Ø 22/150 from steel grade S235 at the interface between the concrete slab and structural steel should ensure a full composite action.

![Figure 1. Overpass structure after erection.](image)

![Figure 2. Structural finite element modelling](image)

Three pairs of reinforced concrete two-column piers of square-shape section provide vertical supports for spans at intermediate points. In addition to transferring vertical loads to the foundations, the piers should resist higher lateral actions caused by potential earthquake events in this low seismic area. The seat-type abutments were constructed separately from the bridge superstructure as the reinforced earth-retaining structure. The bridge superstructure seats on the support stems through pot bearings with teflon sliding surfaces accommodating translational movement. The fixed bearing allowing only rotations but restricting movements is located at the central pier.

2.2. Transformation models and testing results
Curvature and real direction of bearing displacement had to be appropriately implemented into the model. The concrete deck was approximated with shell finite elements, as it can be seen in Figure 2. Seven different thicknesses of elements were adopted for better account of variable deck. Steel girders and cross truss beams as well as support diaphragms were meshed by beam elements, including real eccentricities from bridge slab.
Deflections in mid-spans of steel beams and bearing settlement were measured during loading test. Applied test load should simulate a traffic situation, which produce deformation and internal forces large enough to satisfy given criteria of relevant codes, e.g. STN 73 6209 [2]. Generally, an efficiency relation $\eta$ is specified, as a ratio between elastic experimental values $S_e$ of any observed parameter activated by testing loading and calculated values $S_{cal}$ produced by code ideal load scheme $\eta = S_e / S_{cal}$. In the case of this bridge structure, the eight trucks weighty 28 tons, as testing load were imposed in the second largest span for achieving a deflection $S_e = 17.6$ mm and corresponding $\eta = 0.97$. A level of residual part of deflection $S_r$ produced by irreversible deformation of members and connections is the supplementary important criterion. This remaining deformation has to be fewer than 10 % of total measured values $S_r / S_{tot} < 0.10$. The total measured values are given by the summation $S_{tot} = S_e + S_r$. This additional condition was also satisfied.

In addition, the extra field strain measurements have been executed in centre of the second largest span and at the pier bridge cross-section. For this purpose, nineteen strain gauges were used as illustrated in Figure 3. In vicinity of these segments, the superstructure was modelled with a higher degree of mesh refinement to achieve the well-defined force distribution. Firstly, our numerical finite element model 1 shown in Figure 2 takes into account at the 15 % of the hogging spans adjacent to the support only the steel girder and longitudinal deck reinforcement, as given in EN 1994-2 [3]. Over the internal pier, alternatively model 2 has considered unchanging effective width in hogging region, despite the fact that concrete is subject to cracking for a number of reasons, including direct loading and shrinkage. Stress distributions in Figure 4 indicate that numerical model 2, considering constant concrete effective width of the deck could provide results closer to the measured values. The investigation confirmed evidence that, when transverse reinforcement appropriate to the shear connector spacing is provided, even the cracked slab is able to transfer shear to longitudinal reinforcement at a distance of several slab thicknesses on either side of the steel girders. As conclusion, it can be confirmed that the theoretical model could reproduce suitably real flexural bridge behaviour and the criteria imposed for loading test had been satisfied.

3. Steel-concrete composite expressway bridge

3.1. Structural concept
Two similar parallel continuous composite expressway bridges have two end spans of 21.0 m and 20.0 m as well as intermediate ones of 26.0 or 25.5 m extending over fourteen spans. Thus, the superstructure of every one bridge is 339.0 m long with connected concrete deck 13.55 m wide [4]. The roadways have
a width of 11.25 m, beginning per transit curves and in the second spans proceeding via circular bents of 930 m road radius. Figure 5 shows a typical bridge cross-section consisting of the deck 300 mm thick of concrete class C 35/45 and four built-up girders of I-sections axially 3.0 m spaced. The steel plate girders selected for each bridge have a slender web 1.2 m depth increasing at 1.8 m by haunches next to intermediate supports. Transverse intermediate stiffeners welded to both sides of the web were placed in the third parts of spans allowing webs to develop buckling capacity. The top flange acting with the concrete deck is of the constant 360 mm width and proportioned by varying thickness from 20 to 40 mm to respect bending moment’s differences. The bottom flanges of even 650 mm width remain of constant 40 mm thick. Structural carbon steel S355 grade have been used for steel bridge structural parts. Cross bracings above piers consists of angel’s diagonals and horizontal channels. Other cross girders are located in the thirds of spans. Their spacing is compatible with transverse stiffeners. Headed stud connectors \( \phi 19 \times 150 \text{ mm} \) were used for transmitting longitudinal shear between concrete and steel part of composite structure.

![Figure 5. Cross-section of composite structure for the right express lane.](image1)

![Figure 6. Selected viable finite elements structural modelling details.](image2)

Four trucks as testing load of the average weight around 41.15 tones with variation of 5 % were imposed in designed position, determined by calculation. Measurement of steel beams deflections and bearing settlements were completed in the two edge spans by strains measurements. Electrical resistant gauges were used in the mid-spans and above the adjacent piers.

3.2. Calculation models and testing

Three types of numerical models were used for structural analysis of the bridge. The first one was based on three-dimensional structural finite analysis. This model 1 has approximated concrete deck with plate elements of variable parameters. The steel girders as well as cross beams were meshed by beam elements, as it can be seen in Figure 6. The real structure curvatures and ways of bearing movement were implemented in this model, as well.

In the alternative grid model 2, the main girders consisted of steel-concrete composite sections. The crossbeams and other bracing were connected to the girders, taking into consideration the vertical axe differences of the structural member. The slab strip 2.0 m wide simulated concurrent working of the concrete deck. The slight curvature of the structures was neglected in this model.

The third numerical approximation, specified as the model 3, might represent a quite simple approach. The longitudinally separated steel-concrete main girders without any interaction had to transmit an equivalent part of load applied on the deck, determined by conventional method supposing grid with an infinitely rigid cross-bracing.

Influence of concrete cracking on flexural stiffness modification may be considered by several methods. For this case, in the model 1, the simplified \( \omega \)--method according to our research [5], introducing additional rotation due to crack development in hogging regions has been also used. The lines corresponding to this approach are labelled as model 1--\( \omega \).
Figure 7. Girder deflections in the mid cross-section of the first span for the chosen testing load positions from I. to III.

Several locations of testing loads have been managed during the test. In Figure 7, deflection of all four girders in the middle of the first span caused by three load cases is presented. First load position (I.) represents testing loads unsymmetrically placed within the cross section in both adjacent spans (the first and the second). In the second load case (II.), the heavy weight lorries in the first span stayed in the same position, while the second span was unloaded. During the third load simulation, only the first span was loaded again, but the load was rearranged to a symmetric position within the bridge width.

The resulting deflections of girders illustrated by curves in Figure 7 are showing that the simplest model of separated beams may deliver somewhat inaccurate deformations of cross-sections. The grid model 2 and especially the complex tree-dimensional model 1 can reflect more properly flexural bridge behaviour. Anyway, consideration of concrete cracking did not lead into improvement of the results in all cases.

4. Steel-concrete composite highway bridge

4.1. Structural concept

Roadway deck of this highway bridge is supported by two girders and slim, rather high piers. Because of an angular crossover and arch curvatures of side road approaches, theoretical spans of main girders are 69.0 + 81.0 + 87.0 + 75.0 + 63.0 + 45.0 m. The roadway has a width of 11.5 m between barriers and the total deck width varies from 14.11 to 16.61 meters. Thus, this composite superstructure defines two-lane overcrossing over its whole 422.0 metres length [6]. Figure 8 is showing the bridge layout consisting of the concrete slab and only two built-up plate girders of I-section axially 6.7 m spaced.

The steel plate main composite girders were selected for the bridge having the constant depth of the slender web 4.94 m. Three longitudinal angle stiffeners welded to the outer sides of the webs were placed for meeting slenderness requirements and allowing developing shear capacity as well as bending resistance by preventing local buckling. The top flanges that act with the concrete slab are of the constant 900 mm width and proportioned by varying thickness from 25 to 40 mm. For increasing the flexural strength of cross-sections, the bottom flanges are 1 200 mm width, and vary in the thickness from 30 to 55 mm. The structural steels S355 and S420 as well as S460 have been used for the steel bridge structural parts. Truss cross-frames consisting of horizontal chords and diagonals made of HEB sections provide the lateral stability of the plate-girders and help to distribute vertical loads. The concrete class C 35/45 was used in the slab 250 mm thick with haunches increasing at 450 mm above the girders. Shear stud connectors Ø 19/150 from steel grade S235 should ensure a full composite action.

Solid wall piers of oblong section, both slender and aesthetically appealing provide vertical supports for spans at intermediate points. The seat-type abutments were constructed separately from the bridge superstructure.
4.2. Transformation model and testing under static loads

The concrete deck and steel main girders were approximated with shell finite elements. Varying thicknesses of elements were adopted for better account of variable deck as well as in fact fifteen different girder sections, as shown in Figure 9. Cross truss beams as well as support diaphragms were meshed by member elements.

Figure 8. Bridge structure during erection.  
Figure 9. Bridge finite element modelling.

The efficiency factor as relation $\eta$ between experimental deflection $S_e = 24.6$ mm produced by test loading and calculated values $S_{cal} = 26.0$ mm in the middle of the fifth span developed by code ideal load model LM1 [7], in the case of this bridge structure $\eta = 0.94$ imposed eight trucks, having the total weight of 42.0 tons each. This additional condition of the remaining deformations was also satisfied, since extreme ration $S_e/S_{tot} = 0.086 < 0.10$. This static testing has approved sufficient conformity of the behaviour of the structure with the design calculations.

4.3. Dynamic load testing

The bridge was tested also dynamically through passages of two fully loaded four-axle trucks, with a normal axle spacing of 4.5 m. The gross weight of the vehicle rested near the allowed limit of 42 tons as the maximum legal gross weight of vehicle. The test vehicles were driven at constant speed, along the longitudinal bridge axis and always in the same direction. The tests were begun with a speed of 5 km/h, which was then increased after every passage in steps of 5 to 10 km/h up to the maximum speed 50 km/h. Tests on the undisturbed pavement were repeated with a plank placed on the roadway in the main measurement cross section. The plank was approximately 60 mm thick and 500 mm wide.

Figure 10. Strains of the midpoint of the third 87 m long span under the passage of vehicles traveling from 5 km/h to 50 km/h on the undisturbed pavement.  
Figure 11. Second fundamental mode of bending and torsional oscillation.
Produced strains were measured at the mid-point of the second and third spans, as the bridge main characteristic measurement cross sections. The mechanical vibrations were recorded by two accelerators. The strain time dependent variations, transformed at electrical signals were registered by measurement accuracy apparatus Spider, controlled by a notebook.

From the registered dynamic signals shown as example in Figure 10, the fundamental frequency $f(2) = 1.05$ Hz, very close to theoretical value $f_{theo}(2) = 1.03$ Hz was obtained for the second proper mode of the loaded bridge oscillations, as illustrated in Figure 11. Then logarithmic damping decrement, which required measurement of the magnitude of the first and last strain amplitudes $S_f$ having the same phase, could be identified. Considering the number of periods $i$, the damping was determined from $\nu = \ln (S_{final}/S_i)/i$, and going from 0.052 to 0.064. From the peak value $S_{max}$ of the bridge response during passages of the test vehicles and strain observed under static loading with the same vehicles $S_{stat}$ at Figure 10, the dynamic increment can be found, as $\phi = S_{max}/S_{stat}$. Finally, the bridge critical velocity $c = 26$ km/h was identified as corresponding to the maximum dynamic increment $\phi_{max} = 1.15$.

5. Concluding remarks

Generally, the actual stiffness of the tested structures turned out to be higher than the values predicted analytically. The primary reasons for this were the unintended composite action of non-structural members and partial fixity of supports. Thus, the bridge case studies in this paper might illustrate, that loading tests together with analytical modelling represent very powerful control mechanism. Practically, all relevant differences, defects or even static imperfections can be revealed before putting a bridge into exploitation. Therefore, experimental data and numerical models can present powerful tools for identification of real bridge behaviour, the integrity of the structure evaluation and validation and optimisation of the structural design.

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