NUMERICAL ANALYSIS OF A SOIL-STEEL BRIDGE STRUCTURE

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Abstract. The paper presents a numerical analysis of the soil-steel bridge which was also thoroughly tested under real field loads (during backfilling and under static loads). The comparison of results from calculations and field tests was also presented in the paper. The soil-steel structures are built mostly as bridges located on local roads, but also as the railway viaducts, or even as the highway bridges as well as recently as the ecological objects or tunnels (overpasses and underpasses for animals). The technology of usage of flexible structures made from corrugated steel plates (CSP) is based on the interaction between shell and surrounding soil (backfill) and also takes into consideration the effect of loads arching in soil. The computation model with interface elements can be used to computer simulation of live loads in such type of bridges instead of extremely expensive and time-consuming experimental tests.

Keywords: soil-steel bridge, backfill, corrugated plate, numerical analysis, field load test.

1. Introduction

The main load-carrying element of such structures is an engineering backfill, therefore they are called the soil-steel bridges. The term "soil-steel structure" was first coined in 1976, when the 1st edition of the Ontario Highway Bridge Design Code (OHBDC) in Canada was introduced. Since that time, many scientists and engineers from different countries have been involved in many innovations in the design and construction of soil-steel bridges (Abdel-Sayed, Bakht 1981; Abdel-Sayed *et al.* 1992).

The soil-steel bridge (culvert) structures, consisting of shells of corrugated steel plates and surrounding with well-compacted soil, were first used in the USA. Flexible corrugated-plate designs have been used for bridges and culverts since the 19th century. The technology was patented in 1886 in the USA and since that time, steel corrugated plates find increasingly wider application in transport construction in different parts of the world. However, yet no uniform computing method for dimensioning this kind of shell structures has been developed (Bakht 1981; Pettersson, Sundquist 2003).

The bridge structures, according to their strategic location, the expected long-life as well as the predominant influence of live loads, are very significant. Taking the above into account, one should search the modern design solutions in the bridge engineering connecting fastness of realization, low costs and suitable service durability. The flexible bridges made from corrugated steel plates have all those advantages.

Many various types of flexible structures in relation to the shape of the longitudinal section are well-known. However in the situation when the relatively short time of construction or fast rehabilitation of old bridges is required, the typical structures directly from the producer catalogue can be used. Box, arch, circular, elliptical, arch-circular or pear-shaped structures made from corrugated steel plates (different types of corrugation) are highly suitable as small and/or medium-sized bridges or culverts (Fig. 1). The flexible structures can be founded on the long strip footing made from the reinforced concrete or directly on the ground which must be special designed. For many years, the flexible
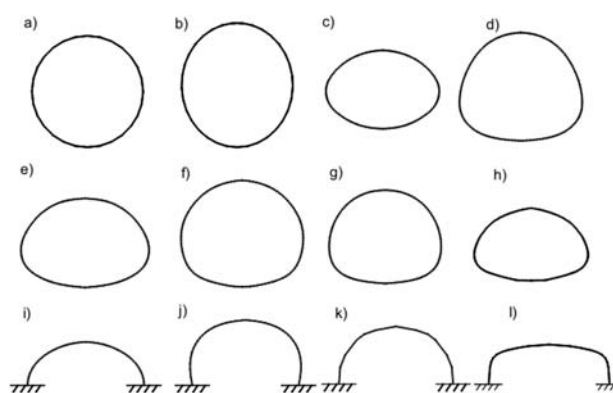

Fig. 1. The shapes of steel flexible structures: a – circular; b – elliptical vertical; c – elliptical horizontal; d – pear; e, f, g, h – arched-circular; i, j, k – arched; l – box

structures are applied to reinforcement and construction of engineering objects, e.g. bridges, viaducts (Fig. 2), tunnels, culverts, footbridges, silos, drainage system, sewage system, ventilating system, casing of conveyor belts as well as for the environmental objects such as the animal crossings situated under and above a highway (Fig. 3).

Fig. 2. Side view on the road viaduct made from steel corrugated plates situated over railway line in Prudnik, Poland

Fig. 3. Top view on the animal crossing situated on the motorway in Poland (Bednarek, Czerepak 2007)

Most current design methods consider pure ring compression as their main design criterion. Ring compression forces are most applicable and most important in cases of structures with high covers. However, as the span increases or soil cover decreases bending moments, displacements and stresses become a more significant design consideration.

For the soil-steel structure (e.g. pipes, culverts, bridges) 3 main design criteria are usually used.

1. The deflection criteria. The most commonly used deflection equation is the well-known Iowa formula, developed by Spangler and Martson for computing the change in the horizontal diameter of flexible steel (or plastic) culverts. This formula consists of the controversial modulus of soil reaction. Spangler postulated that pressures distributions are proportional to movement (Vaslestad 1990). The assumption of Iowa deflection formula may be summarized as follows:

- the vertical load is distributed uniformly over the width of the structure;
- the reaction on the bottom of the structure is equal to the vertical load and is distributed uniformly over the bedding of the structure;
- the horizontal pressure is distributed parabolically over the middle 100° of the structure, and the max unit pressure is a result of the modulus of passive resistance and 1/2 of the horizontal deflection of the structure.

2. The thrust criteria – compression loads for the structure wall. The thrust is limited to a value that will not cause seam failure or wall material yielding. The ring compression theory is used. This theory states that the structure wall should be designed to resist the compression stresses and deflections produced by a hydrostatic soil pressure equal to the overburden pressure. The distribution of soil pressure according to the ring compression theory what was developed by White and Layer for 3 different shapes (Beben 2005).

3. The buckling criteria. Elastic buckling is originated as a local bugle and may occur at the crown or at other locations, depending on where critical combination of force, bending moment imperfection and residual stresses 1st develops (Beben 2005; Vaslestad 1990).

On basis above-mentioned of 3 conventional design criteria following methods were developed: The AISI method prepared by the American Iron and Steel Institute, the ASTM method (prepared by the American Society for Testing Materials), the AASHTO method prepared by the American Association of State Highway and Transportation Officials, the ARMTEC super-span method (prepared by Armtec International from Canada), The CANDE program (prepared by the Federal Highway Administration (FHWA)), The SCI method (prepared by Duncan, M.), The UBC culvert design procedure (it is improvement of SCI method), the OHBDC method (prepared by the Ontario Highway Bridge Design Code), and the CHBDC method (prepared by the Canadian Highway Bridge Design Code).

The above-mentioned conventional methods are very conservative in comparison to results received from the experimental tests and the numerical calculations (Abdel-Sayed and Bakht 1981; Abdel-Sayed *et al.* 1992; Vaslestad 1990).

2. Analysis of soil-steel bridge

2.1. General notes

For the most part, current design methods for soil-steel bridge structures are based on experience rather than a viable analytical model. This is because a reasonable analytical model is quite complicated. These complications are due to the soil-steel structure interaction phenomena requiring both soil and structure to be considered as structural elements and one can not simply assume the loads acting on the structure (Beben 2005; Vaslestad 1990).

The finite element method (FEM) and finite difference method (FDM) are widely used to analyses the stress-

es and displacements in the soil-steel system. However, the data derived from the computational analyses are no more accurate than the process it is simulating and the validity of the soil models and the soil parameters incorporated in the formulation.

Up to now, the calculations results obtained with use of applied computational models in comparison to the tests results conducted on real objects are insufficiently accurate (Katona 1978; McCavour *et al.* 1998; Mohammed *et al.* 2002; Vaslestad 1990). Therefore, the materials characteristics for soil, steel structure and contact layers (between various materials) should be assumed as the most real.

2.2. Description of the analysed bridge

In the longitudinal section, the analysed road bridge has a static structure of a single span steel shell connected rigidly with a continuous footing of effective length $L = 12.315$ m (Fig. 4). The shell is supported by means of unequal steel leg channels resting on 2 reinforced concrete continuous footings.

The basic shell of the structure was reinforced in 3 places, i.e. the crown and in 2 corners at the continuous footings from the side of soil at both sides of the object

by means of additional sheets of corrugated plate, the so-called ribs (in the crown the reinforcing is continuous and in haunches the pitch is 380 mm) in order to assure greater transversal stiffness of the bridge span (Fig. 4). The load bearing structure was constructed as a shell assembled from the sheets of corrugated plate of corrugation depth of $h = 140$ mm, pitch of $a = 380$ mm, and plate thickness of $t = 7.10$ mm, connected together using high strength bolts, covered with the layers of soil (about 0.20–0.30 m thick) properly compacted (in the standard Proctor scale $I_D = 95\%$ for the soil connected directly with the steel structure and $I_D = 98\%$ for the remaining part of the backfill), allowing pavement to be laid on broken stone base. The total height of span structure (i.e. the plate corrugation height) is $h = 140$ mm. The width of the bridge shell at the top is $b_t = 12.915$ m, and at the bottom $b_b = 20.574$ m. In the plan view, the object is situated perpendicularly to the river current, and the vertical rise amounts to $h_0 = 3.555$ m (Fig. 4). Basic dimensions of the bridge are shown in Fig. 4, whereas the finished bridge object prepared for final load tests is presented in Fig. 5.
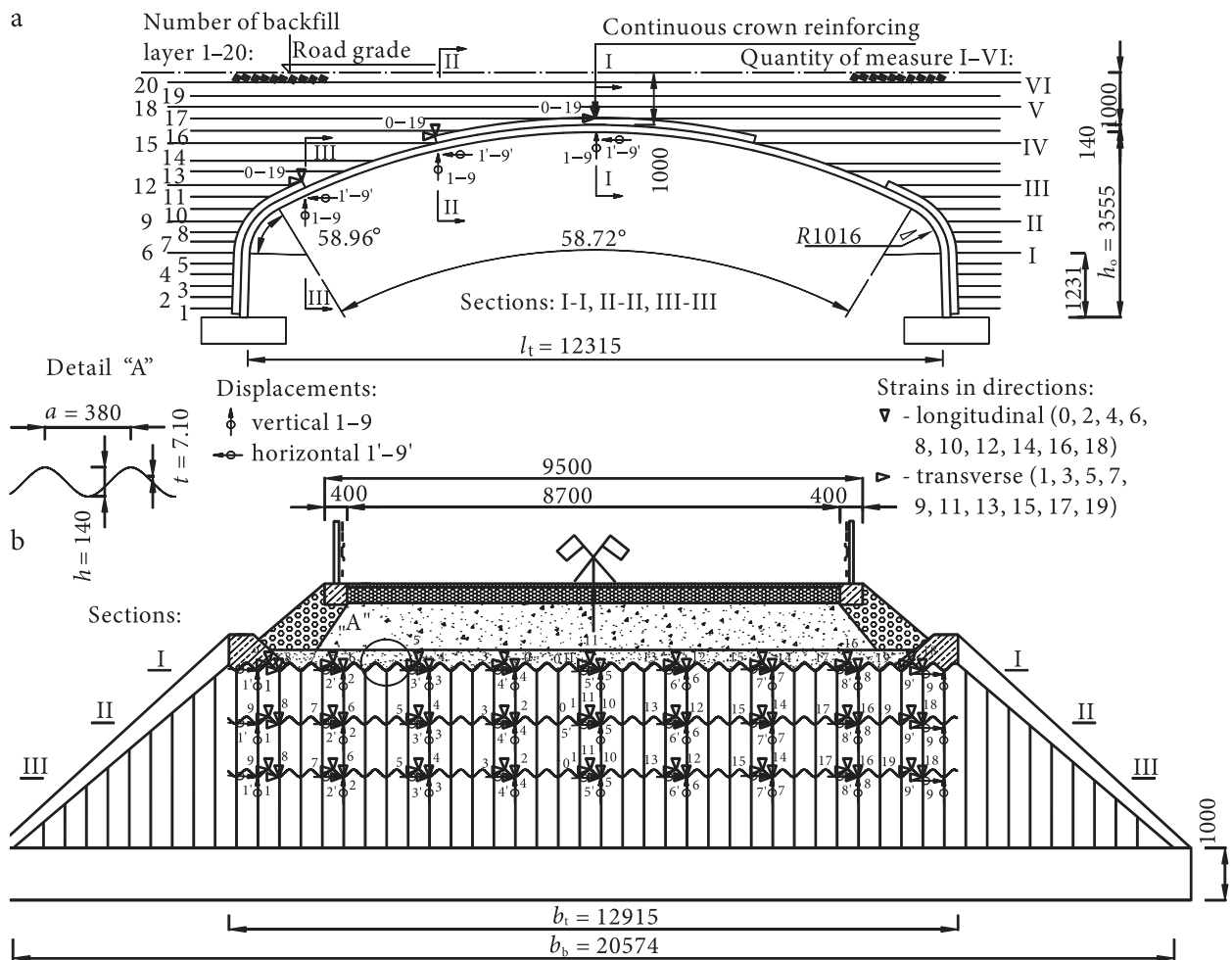

Fig. 4. Road bridge in Gimån (Sweden), made from corrugated plates: a – geometry of longitudinal section; b – cross-section I–I localization of dial gages and extensometers served measuring of vertical (horizontal) displacements and strains in longitudinal (transversal) directions, respectively (measurements in mm)

Fig. 5. Side view on the soil-steel bridge made from corrugated plates (in Sweden) (Manko, Beben 2008)

2.3. Numerical solution

2.3.1. General assumptions

In resent years to obtain solutions of many geotechnical problems the FEM (FDM) has been widely used (the two- and three-dimensional model analysis). The soil and steel structure walls are replaced by discrete elements interconnected at nodal points and these elements are assigned the material properties or the original continuum.

The significant characteristics of a soil-steel bridge structures that can be handled by the method are: geometry of structure; soil boundary conditions; incremental construction method; material properties (non-linearity, stress dependent soil behaviour); interface layers (or elements) between 2 various materials (e.g. soil and steel) which will obtain the interaction between them.

The considerable differences between the computed values and the measured ones lie mainly in the computations and in the fact that it is extremely difficult to determine, using the adopted model, the extent of the interaction between the steel shell structure and the surrounding backfill. Difficulties were encountered when determining the extent of interacts between the steel shell structure and the surrounding soil in carrying service loads and when modelling the interface between the corrugated steel plate structure and the backfill.

The calculations of the analysed bridge done by AIL (Atlantic Industries Limited) were verified for the same (or similar) assumptions using the computer program Robot Millennium and similar values were obtained. Therefore, it was decided to conduct own calculations on real assumptions in FLAC 2D program ver 3.4 from the non-linear contact elements of interface type. This program is based on FDM, which permits of the realization of the selected problems concerning statics and dynamics of the structure for the plane flexible systems.

The soil is modelled as elastic-plastic model (criterion Coulomb-Mohr), with linear modulus variations with depth. Modulus variation $E(z) = E_0 + mz$ is defined using surface modulus E_0 and modulus gradient m. 2 data parameters used to modelled for 95% – the 1st (and 98% – the 2nd) Normal Proctor are following: Poisson's ratio $v_g = 0.20$ (0.0); cohesion $c_g = 0.0$ (0.0), friction angle $\varphi_g = 43°$ (35°); gradient $m_g = 3.8$ MPa/m; $E_{0g} = 20$ MPa and unit weight of soil $\gamma_g = 20$ kN/m³, dilation angle $\psi = 0.0°$ (0.0°), failure ratio $R_f = 0.96$ (0.85). The corrugated steel structure of box culvert type was modelled as bilinear elastic with material constants of: initial Young's modulus $E_{1s} = 207$ GPa; secondary Young's modulus $E_{2s} = 12$ GPa; Poisson's ratio $v_s = 0.30$; yield stress $\sigma_{ys} = 300$ MPa, plate thickness $t = 7.1 \times 10^{-3}$ m, moment of inertia $I = 122\,638$ mm⁴/mm. The asphalt material was considered linear elastic with Young's modulus $E = 6.9$ GPa and Poisson's ratio $v = 0.41$.

2.3.2. Interface elements

There are several instances in geomechanics in which it is desirable to represent planes on which sliding or separation can occur. The additionally specific interface elements on the point of contact of different materials should be applied to formulation of these problems.

The formulation of the growth of structure response facilitates modelling the non-linear behaviours, such as how the characteristics of soil and structure: the stress-strain ($\sigma - \varepsilon$) as well as interaction of the steel shell-soil system with regard to the contact layer on point of contact both materials. One should provide interface elements that are characterized by Coulomb sliding and or tensile separation.

Interfaces have the properties of friction, cohesion, dilation, normal and shear stiffness, and tensile strength. Although there is no restriction on the number of interfaces or the complexity of their intersections, it is generally not reasonable to model more than a few simple interfaces with the FLAC program because it is awkward to specify the complicated interface geometry.

An interface element is represented as a normal (k_n) and shear stiffness (k_s) between 2 planes, which may contact one another, as was shown in Fig. 6. For it description uses contact logic for either side of the interface which is similar in nature to that which is employed in the distinct element method (Beben 2005).

The non-linear hyperbolic interface element properties are: normal stiffness $k_n = 2.6 \times 10^8$ kNm⁻³, shear stiffness $k_s = 2.6 \times 10^8$ kNm⁻³, friction angle $\varphi = 30°$, cohesive strength $c = 1.0$ kPa, failure ratio $R_f = 0.85$, tensile strength $T = 0$.

The code includes a list of the grid points (i, j) that lie on each side of any particular surface. Each point is taken, in turn, and checked for contact with its closest neighbouring point on the opposite side of the interface element. Referring to Fig. 6, grid point N is checked for contact on the segment between M and P. If contact is detected, the normal, n, to the contact, N, is computed, and a "length" L, defined for the contact along the interface belonging to N, where L is equal to 1/2 the distance to the nearest grid point to the left plus 1/2 the distance to the nearest grid point to the right, irrespective of whether the neighbouring grid point is on the same side of the interface or on the
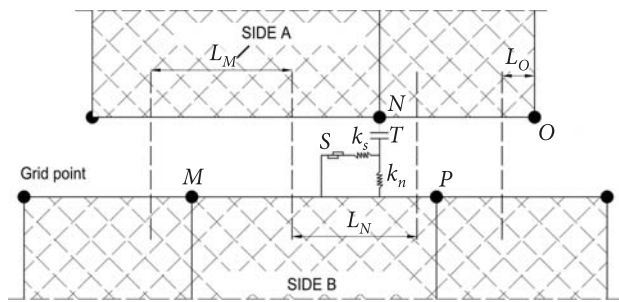
Fig. 6. An interface element represented by sides A and B, connected by shear (k_s) and normal (k_n) stiffness springs: S – slider; T – tensile strength; L_N – length associated with grid point N; L_M – length associated with grid point M; L_0 – length associated with grid point O; – – – denotes limits for joints segments (placed halfway petween adjacent grid points)

opposite side. In this way, the entire joint is divided into contiguous segments, each controlled by a grid point.

During each time step, the velocity, $\mathring{u}_i$, of each grid point is determined. Since the units of velocity are displacement per time step and the calculation time step has been scaled to unity to speed convergence, then the incremental displacement for any given time step is amount $\Delta u_i \equiv \mathring{u}_i$. Next the incremental relative displacement vector at the contact point is resolved into the normal and shear directions, and total normal and shear forces are determined at calculation time ($t + \Delta t$) by Eq (1) and (2):

$$F_n^{(t+\Delta t)} = F_n^{(t)} - k_n \Delta u_n^{\left(t+\frac{1}{2}\Delta t\right)} L, \qquad (1)$$

$$F_s^{(t+\Delta t)} = F_s^{(t)} - k_s \Delta u_s^{\left(t+\frac{1}{2}\Delta t\right)} L, \qquad (2)$$

where $F_n^{(t + \Delta t)}$ – total normal force at time ($t + \Delta t$), kNm; $F_s^{(t + \Delta t)}$ – total shear force at time ($t + \Delta t$), kNm; k_n – the stiffness, kNm^{-3}; k_s – the stiffness, kNm^{-3}; Δu_n – incremental relative normal displacement, m; Δu_s – incremental relative shear displacement, m; L – effective contact length, m.

Next the Coulomb shear-strength criterion limits the shear force by the relation (3):

$$F_{s_{max}} = cL + tg\varphi F_n, \qquad (3)$$

where $F_{s_{max}}$ – Coulomb shear-strength criterion limits for the shear force, kNm; c – cohesion along the interface, kPa; L – effective contact length (Fig. 6), m; φ – friction angle of interface surfaces, (°); F_n – total normal force, kNm.

### 2.3.3. Analysis during backfilling

The two-dimensional computation model for the "combined" soil-steel structure was assumed. The computation analysis was executed in the range of linear statics with non-linear contacts elements of interface type. The loads

in the form of backfill layers were applied as evenly spreading on width of 1.00 m. The analysis was conducted for 6 selected numbers of backfill layers (Fig. 4) which were the sand and broken stone alternatively what corresponded with layers (6, 9, 12, 15, 18, 20 – it is number of backfill layers), for which experimental field tests were also executed (Beben 2005; Manko, Beben 2005b).

The two-dimensional analyses are performed step by step, beginning with the structure resting on its foundation with backfill. The placement of the 1st layer of backfill alongside the culvert is modelled by adding the 1st layer of soil elements to the finite difference (element) mesh. At the same time, loads are applied representing the weights of the added elements. Through their interaction, the soil elements load the structure. Subsequent steps of the analyses are performed in the same way, adding one layer of elements at the time, which simulates the process of backfilling around and over the shell structure. After the final layer of fill has been placed over the top of the structure, loads are applied to the surface of the fill to simulate vehicular traffic loads (Manko, Beben 2005c). Some graphs of max deflections are presented in Fig. 7 and Table 1.
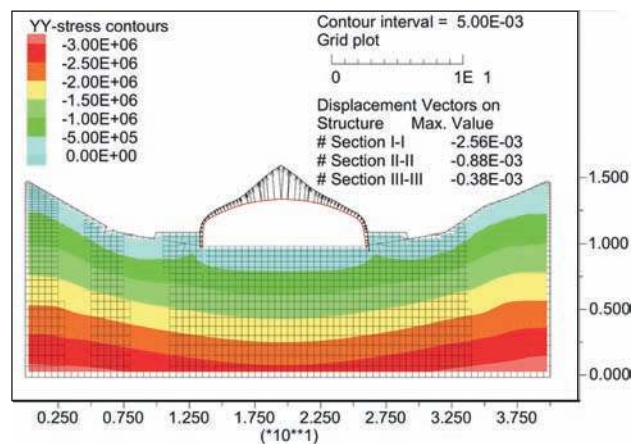
9 of soil layers
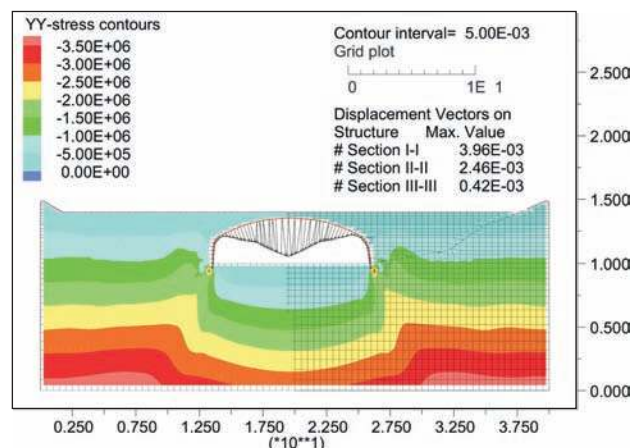
20 of soil layers

Fig. 7. Graphs of max vertical displacements in steel shell structure obtained from backfilling and compacting the backfill

Table 1. The values of max vertical displacements received from measurements and calculations (10^{-3} m) in steel shell structure in analysed sections during the soil compaction

Number of tested backfill layer	Sections											
	I–I				II–II				III–III			
	f_{vm} (10^{-3} m)	f_{vc} (10^{-3} m)	$\dfrac{f_{vm}-f_{vc}}{f_{vm}}$, %	$\dfrac{f_{vm}}{f_{vc}}$	f_{vm} (10^{-3} m)	f_{vc} (10^{-3} m)	$\dfrac{f_{vm}-f_{vc}}{f_{vm}}$, %	$\dfrac{f_{vm}}{f_{vc}}$	f_{vm} (10^{-3} m)	f_{vc} (10^{-3} m)	$\dfrac{f_{vm}-f_{vc}}{f_{vm}}$, %	$\dfrac{f_{vm}}{f_{vc}}$
I	−1.37	−1.49	8.75	0.92	0.67	0.64	4.47	1,04	1.03	1.13	−9.71	0.91
II	−2.34	−2.56	9.41	0.91	−0.83	−0.88	6.02	0.94	1.07	1.15	7.47	0.93
III	−2.57	−2.75	7.00	0.93	−1.20	−1.33	10.83	0.90	1.02	1.07	4.90	0.95
IV	2.65	2.87	−8.30	0.92	1.98	1.85	6.56	1.07	**1.17**	1.31	11.96	0.89
V	3.56	3.82	−7.31	0.93	**2.50**	**2.62**	−4.80	0.95	1.04	1.14	9.61	0.91
VI	**3.72**	**3.96**	−6.45	0.94	**2.62**	**2.46**	4.58	1.04	1.15	1.22	−6.08	0.94

Notes: Vertical displacements respectively: measured f_{vm}, and calculated f_{vc}. Bold means the max value of displacements.

2.3.4. Static analysis

The new finite differences procedures (FDM) used for the soil-structures interaction (during static analyses) are based on the techniques for modelling soil stress-strain behaviour. This is a semi-analytic procedure based on the use of a two-dimensional different (element) finite mesh and Fourier integrals to treat the variations in load and response in the axial direction. This approach leads to a harmonic decomposition in the axial direction and computationally efficient compared with conventional three-dimensional formulations. However, it is based on the principle of superposition and requires linear material behaviour. Furthermore, the Fourier integrals imply modelling the culvert as infinitely long.

The original formulation of Moore and Brachman (Moore, Brachman 1994) has been modified to incorporate orthotropic shell elements, which are based on the harmonic axisymmetric shell elements of Rotter and Jumikis (Rotter, Jumikis 1998) but have been modified in 2 ways. First, the harmonic formulation was redeveloped with the Cartesian coordinate system, permitting use in problems with prismatic geometry, like the steel culvert. Second, the harmonic formulation was adapted for use in Fourier integral instead of Fourier series analysis. This permits consideration of just one set of applied loads in the axial direction of the bridge (i.e. one truck) instead of periodic loading as required when Fourier series are used (Beben 2005). Some graphs of max normal stresses are presented in Fig. 8 and Table 2.

2.4. Experimental test

The main objective of the experimental studies is assessment of behaviour of the soil-steel bridge structure under a known load to validate the assumptions made in the static structural analysis and in the test live-load as well as to determine the such system's actual load capacity and the contribution of soil consolidation to the reduction of the impact of vehicle wheels on the steel shell structure. Mainly the stiffness of the corrugated plates in the span structure is to be assessed and the shell's width interacting in carrying loads (in particular, the transverse distribution of the load among the individual part of corrugations) should be determined. The conclusions drawn from the field tests are a bearing on the acceptance of the bridge for normal service under the properly national requirements and served as the basis for the post-construction recommendations.

Because not many soil-steel bridges (with a relatively large-span) are built in Europe, the original (routine) range of acceptance tests should be extended to comprehensive testing under a dynamic load and under a normal service load. Many field load test studies concerning different types of the flexible structures and also various kinds of applied loads (for example, during construction of such structures, static loads, dynamic loads and normal service) are described in papers (Beben, Manko 2005a, b, c; 2008).

This paper shows the selected experimental results obtained during backfilling and during static load tests which are compared with numerical data (Figs 9, 10 and Tables 1, 2).

3. Conclusions

Properly constructed soil-steel bridge structures are viable, economical substitutes for conventional steel and concrete bridges (and culverts). Design confidence in traditional structures is obviously greater than soil-steel structures due to variety of reasons. The most obvious is the large difference in the numbers of two types of structures built.

Using the FDM or FEM to analysis of estimation of interaction between backfill and steel shell structure made from corrugation plates depended on creating the computation model of load carrying system by suitable discreteization (meshing) to the finite differences (elements) with the non-linear contact elements. All characteristic components appearing in such type of bridges as well as their interaction were considered in the computation model by application of interface elements.

As the effect of executed calculations by the FDM and the experimental research on the real objects was affirmed, that for engineering aims the steel-soil bridge structures analysis is possible to carry out in the plane state of strains (the 2D analysis) with the contact elements of the interface type between steel shell and backfill. In some special cases, the calculations were possible also to execute in the three-dimensional space 3D with the aim of more detailed analysis.
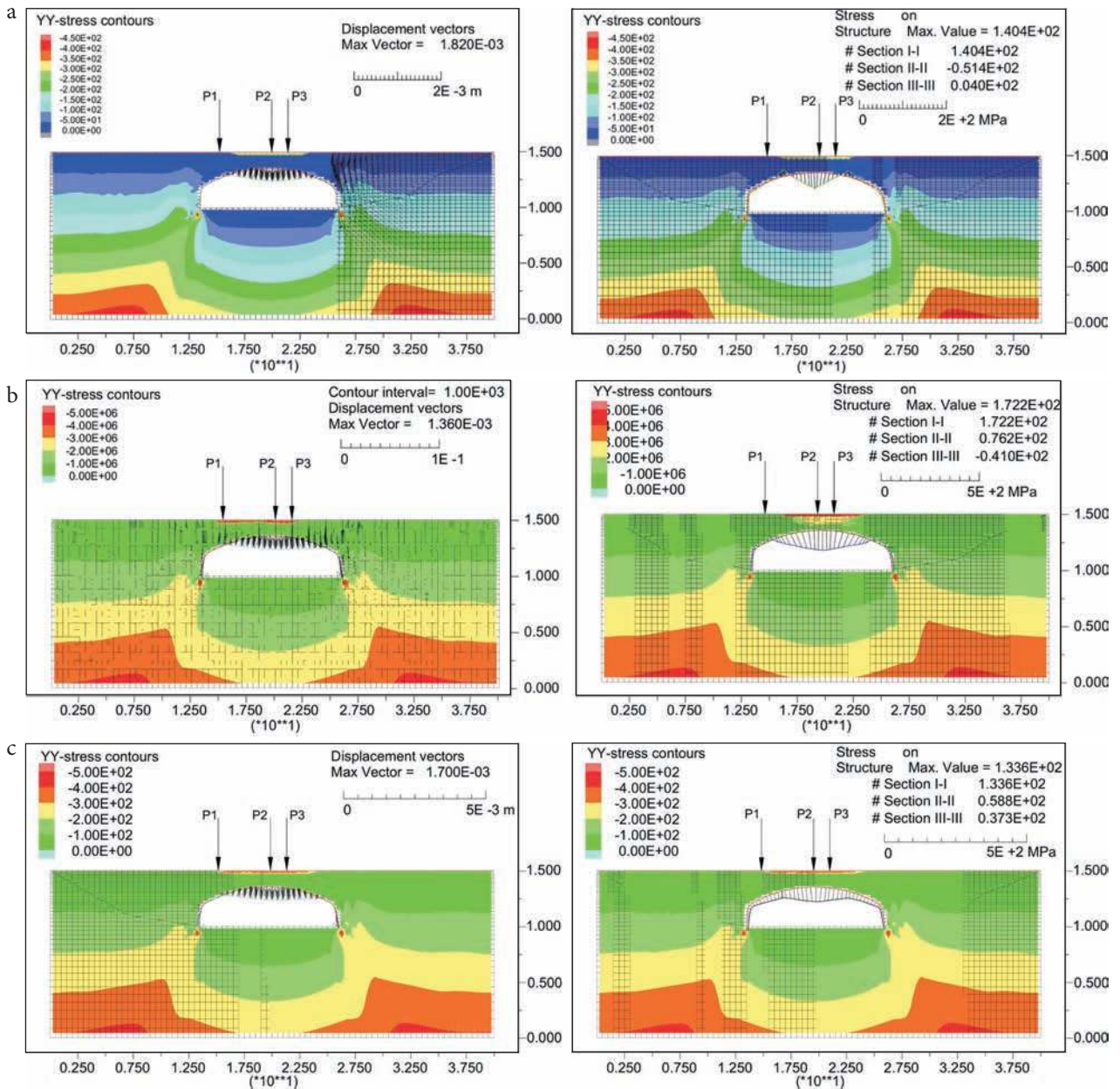
Fig. 8. Graphs of max deflections and normal stresses in shell structure obtained from 3 load schemes: a – I; b – II; c – III

Table 2. Max measured and calculated strain values (10^{-6}) in longitudinal direction in selected 3 sections of steel shell structure obtained at bottom and top of corrugation under static load

Load schemes	Cross-sections											
	I–I				II–II				III–III			
	ε_{xm} (10^{-6})	ε_{xc} (10^{-6})	$\dfrac{\varepsilon_{xm}-\varepsilon_{xc}}{\varepsilon_{xm}}$, %	$\dfrac{\varepsilon_{xm}}{\varepsilon_{xc}}$	ε_{xm} (10^{-6})	ε_{xc} (10^{-6})	$\dfrac{\varepsilon_{xm}-\varepsilon_{xc}}{\varepsilon_{xm}}$, %	$\dfrac{\varepsilon_{xm}}{\varepsilon_{xc}}$	ε_{xm} (10^{-6})	ε_{xc} (10^{-6})	$\dfrac{\varepsilon_{xm}-\varepsilon_{xc}}{\varepsilon_{xm}}$, %	$\dfrac{\varepsilon_{xm}}{\varepsilon_{xc}}$
at bottom edges												
I	333	370	−11.11	0.90	−651	−632	2.92	1.03	259	276	−6.56	0.94
II	800	840	−5.00	0.95	−650	−658	1.23	0.98	303	313	−3.30	0.97
III	376	395	−5.05	0.95	−521	−548	5.18	0.95	321	345	−7.47	0.93
at top edges												
I	374	413	−10.42	0.91	299	323	−8.03	0.92	647	621	4.02	1.04
II	562	615	−9.43	0.91	680	712	−4.71	0.95	630	651	−3.33	0.97
III	355	380	−7.04	0.93	333	358	−7.51	0.93	523	577	−10.32	0.91

Notes: measured ε_{xm} and calculated ε_{xc} strains at top and bottom of corrugations.
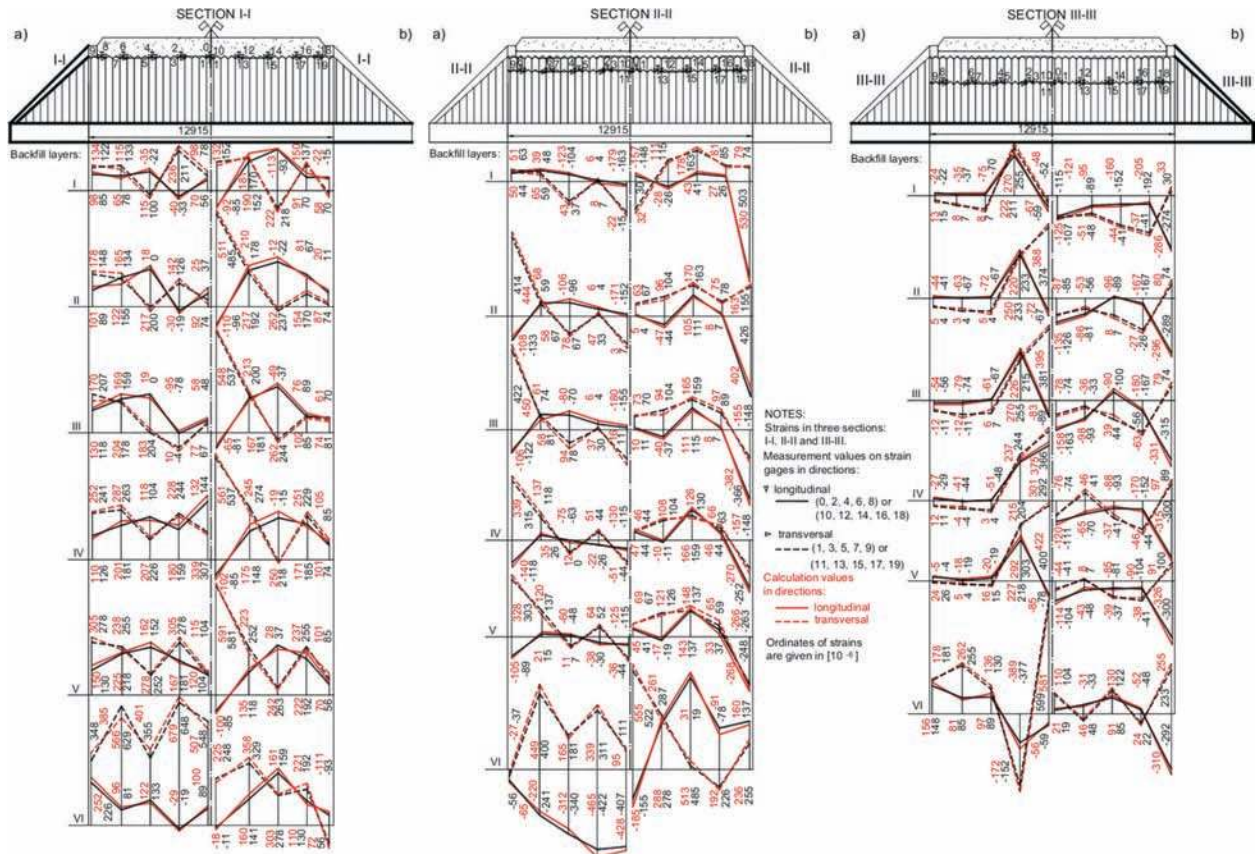
Fig. 9. Graphs of max strains measured and calculated in selected points of steel shell structure in transversal direction of span during compaction and consolidation of each layer of soil in 3 sections I–I, II–II and III–III on: a – top of corrugations; b – bottom of corrugations

However, it requires using the advanced and enough complex computer equipment. The modelling of soil as elastic-plastic (Coulomb-Mohr model) is recommended or as elastic-plastic material with reinforcement, whereas the steel shell as the bilinear elastic material is possible to analyse. The contact layers of interface type with non-linear proprieties should be considered between soil and steel elements.

The behaviour of the flexible structure is dependent on a large degree on their interaction with surrounding soil backfill, which restrains the tendency of the sides of the structures to flex outward and greatly increases the load-carrying capacity as compared with that of a free-standing structure. It is this aspect of their behaviour that makes the use of soil-structure interaction analyses, with simulation of behaviour of both soil backfill and steel shell structure, absolutely essential to provide a realistic basis for design. The advantage of steel flexible bridge structures made from CSP is that their interaction with the surrounding soil (usually native soil) can be exploited.

After a detailed experimental verification and analysis of obtained results, this computation model can be used for computer simulation of service loads in such type of bridges. Such analysis can be applied instead of extremely expensive and time-consuming experimental tests carried out on the individual real objects.

Architectonically the corrugated plate bridge structures are not less attractive than the traditional prefab-ricated concrete, steel plate girder or composite (steel + concrete) structures. In addition, they lend themselves to different finishing treatments as a result of which their aesthetics can be greatly enhanced so that the structures will perfectly blend with their surroundings.

In order to grow carrying capacities of such structures and decrease the shell wall deformations as well as strains, additional reinforcements are applied (especially in the large-span objects). It is made with use of: the steel stiffening ribs, the concrete or steel beams, the reinforced concrete slabs, geofabric (and geosynthetic) as the reinforcement of soil, and also by application of the circumferential reinforced concrete collar as the reinforcement of the structures ends.

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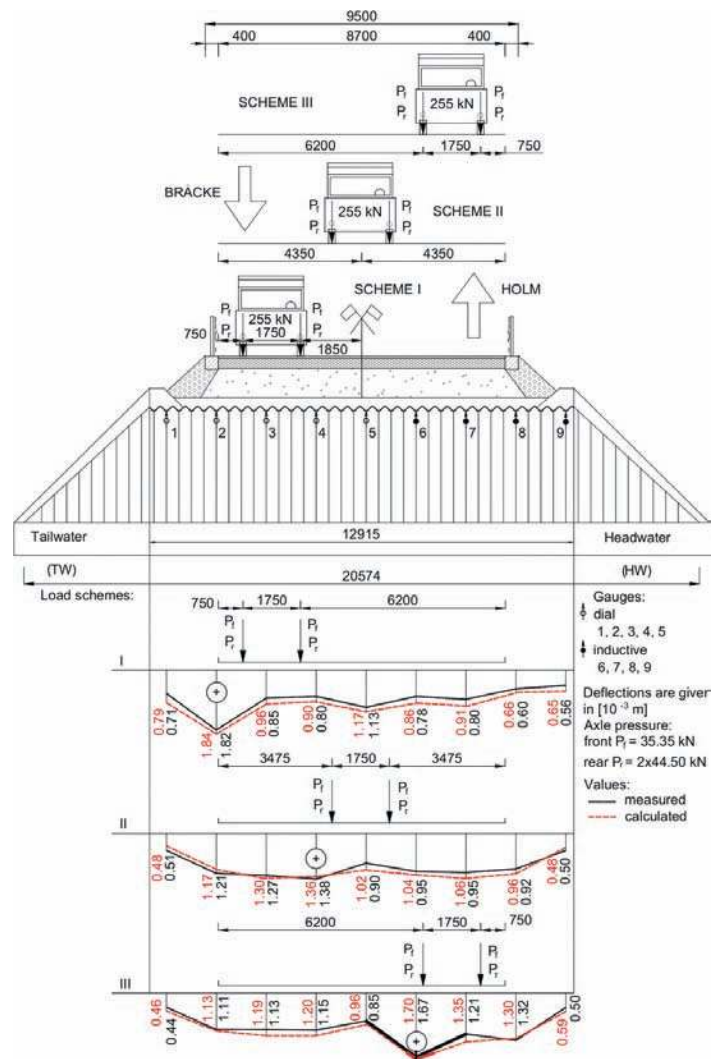
Fig. 10. Graphs of max deflections obtained from measurements and calculations in transversal direction of the span for 3 test load schemes

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