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BEHAVIOUR OF STEEL-SOIL BRIDGE STRUCTURE MADE OF CORRUGATED PLATES UNDER FIELD LOAD TESTS. PART I: STATIC RESEARCH

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The way in which a new road bridge made from *Super Cor* steel plates was tested is described and the test results, for three static load schemes in which one ballasting vehicle (a *Scania* truck) was used as the load, are presented. The tested bridge has a box structure and it is located on the Gimån River in Gimån, Sweden on the Bråcke – Holm road. The bridge has an effective span of 12.315 m and a clear height of 3.555 m. The steel shell of the span is founded on two reinforced concrete continuous foundations. The average measured displacements and unit strains (normal stresses) in selected points and elements of the steel shell structure were found to be much smaller than the ones calculated for the same load. The conclusions drawn from this research can be useful for assessing the behaviour of such steel shells and their interaction with the surrounding backfill. Since such steel–soil structures are used more and more often for small and medium-sized bridges on road and railway lines in Poland and in the world, the conclusions from the static load tests can be generalized to a whole class of similar bridge structures.

1. Introduction

The way in which a single-span, flexible-structure bridge made from *Super Cor SC-56B* corrugated steel plates, located in Gimån on the Bråcke – Holm road (no. 716) in Sweden, was tested is described, and the results of the tests and static-strength calculations that serve as the basis for determining the quality and durability of the bridge and accepted it for normal service in view of its quite large effective span and prototypical character (the first bridge of this type in Scandinavia) are presented [1].

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The primary aim of the static (and dynamic) load tests was to determine the effort of the structural components of the bridge and to assess the workmanship and the performance of the shell structure under a known load in order to verify the assumptions made in the static calculations and analyses of the span and in the test load program and to determine the actual load-carrying capacity of the bridge. In particular the actual rigidity of the corrugated plates in the arch structure was to be evaluated and the width of the deck plate (corrugated plate) interact with the soil in carrying service loads and the transverse distribution of the loads among the individual ribs (folds) were to be determined. Measurements were performed in three span cross-sections along the length of the bridge under a symmetric and asymmetric (relative to the longitudinal axis of the bridge) load. Three static load schemes, involving one truck positioned at half of the effective span length, were considered [1].

The conclusions drawn from the research had a bearing on the acceptance of the bridge for normal service under the specified load and could serve as the basis for post-construction recommendations. The tests described here were the acceptance tests required by the relevant bridge codes: Polish PN-77/S-10040, PN-82/S-10052, PN-85/S-10030 and PN-89/S-10050, Swedish Vägverkert – VU 94: Vägutformning 94, Borlänge 1994, Vägverket – 1990:11: Hydraulisk dimensionering, Borlänge 1994, Vägverket – 1994:15: Jords hållfasthets – och deformationsgenskaper and Borlänge 1994 and American AISI – American Iron and Steel Institute: Handbook of Steel Drainage & Highway Construction Products, Fifth Edition, Chicago 1994 and AASHTO – American Association of State Highway and Transportation Officials, Standard Specifications for Highway Bridges, New York 2002. They were also to prove that the span structure and the continuous foundations (supports) had been properly made and their results were to be the basis for a decision allowing the bridge to be put into normal service under the load specified by the relevant Swedish standards (corresponding to the Polish class B load of 400 kN).

Considering the fact that the bridge was of strategic importance for the road network in northern Sweden, and was to carry quite heavy loads and that not many such bridges had been built for in Europe, the initial (routine) range of acceptance tests was extended to cover dynamic impact tests [2], tests of the steel shell during backfilling (possible loss of stability) [3, 4], and the so-called service test under an actual load [1]. The comprehensive and thorough tests, the detailed analysis of displacements and strains, and the conclusions drawn from the research can be useful in engineering practice, particularly in the field of test loads and check and acceptance tests of steel-soil road (or railway) bridges made from corrugated or flat plates or for developing acceptance guidelines, considering that the relevant regulations currently in force do not cover bridges of this type [2–14].

The paper presents tests of road bridge which the reinforcement of shell was made from corrugated steel plates with an application of new localization (Fig. 1) and relatively small height cover of soil over the shell in comparison to different bridge structures of this type [4, 6, 7].

2. Description of bridge structure

In the cross-section the tested bridge is a static structure having the form of a single-span, rigidly supported "box" (*box culvert* according to terminology of the manufacturer – *Atlantic Industries Limited*, Canada) – Fig. 1a. The effective span length is $l_t = 12.315$ m. The superstructure of the bridge is a shell

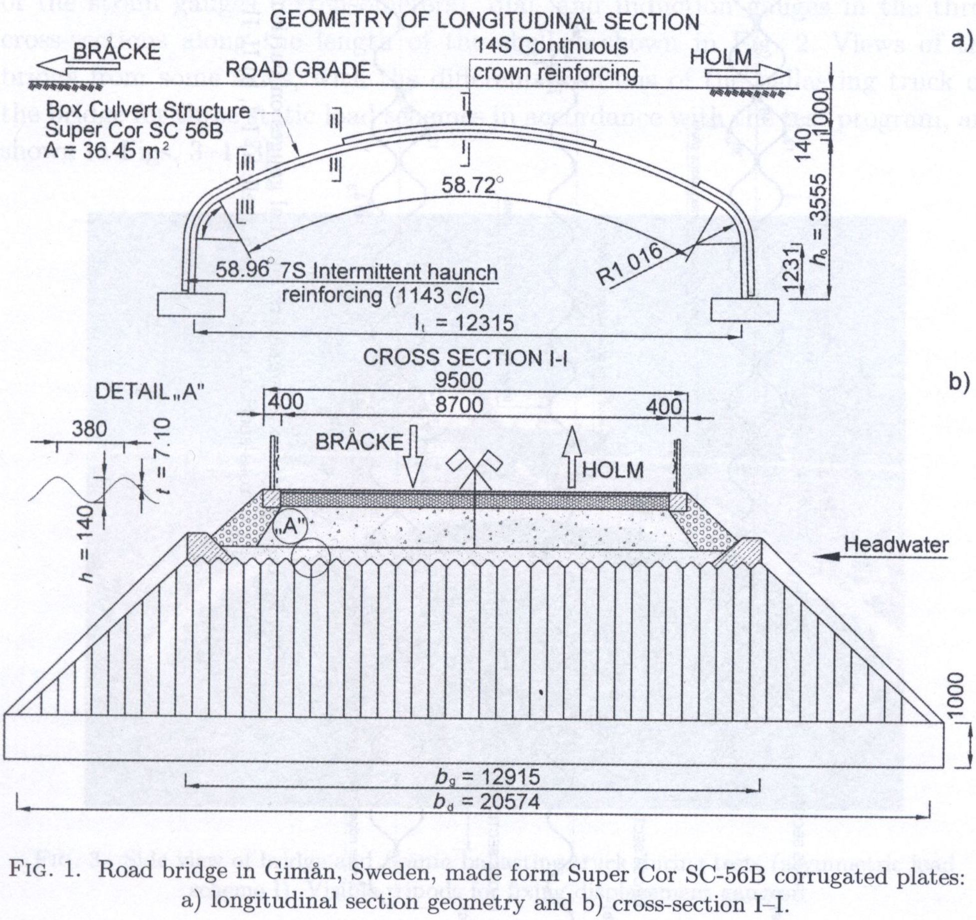

FIG. 1. Road bridge in Gimån, Sweden, made form Super Cor SC-56B corrugated plates: a) longitudinal section geometry and b) cross-section I–I.

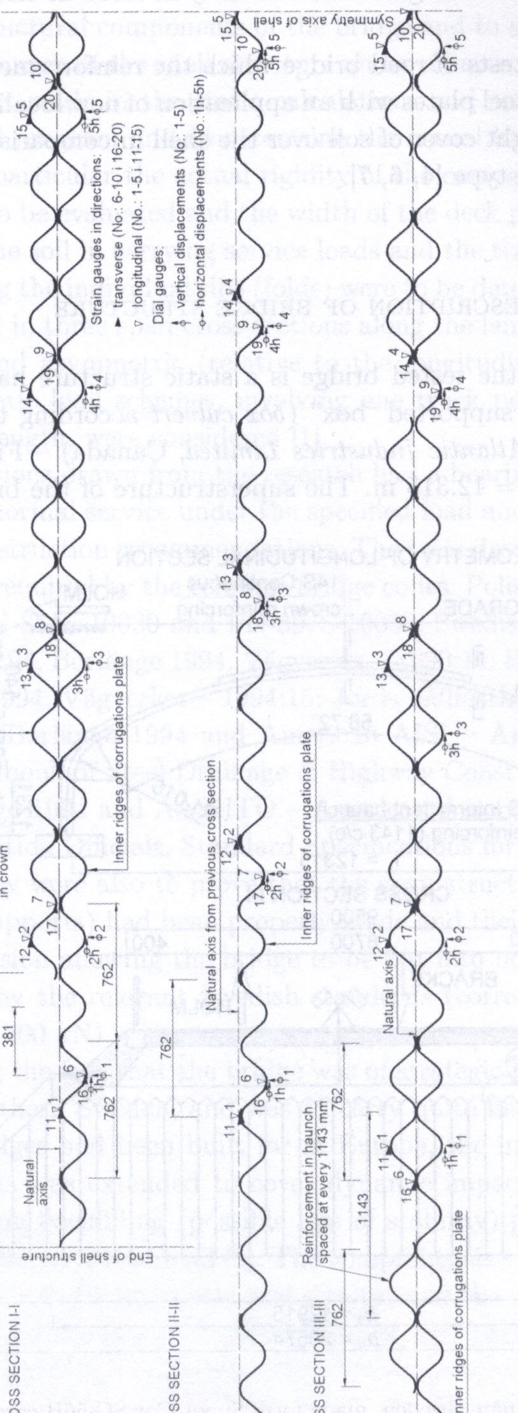

FIG. 2. Location of extensometers and dial gauges on particular corrugated plates in three cross-sections of the steel shell: a) I–I, b) II–II and c) III–III.

made from 140×380 mm corrugated. The steel plates joined together across the spanby high tensile bolts M20 (class SB 8.8) tightened with a twisting moment of 350–400 Nm. The shell is founded, by means of steel uneven-armed channel sections, on two reinforced concrete continuous footings. The span structure was reinforced in three places, i.e., in the crown and in the two haunches (at the foundations) on the soil side on both sides of the bridge, to increase the rigidity of the superstructure. The shell was earthed up with 0.20–0.30 m thick layers of permeable soil compacted to $I_D = 0.95$ (on the Proctor scale) for the soil being in direct contact with the steel shell and to $I_D = 0.98$ for the other backfill, whereby a pavement could be laid on a broken stone subgrade. The total height of the superstructure (the height of the corrugations) is $h = 140$ mm. The width of the shell is $b_t = 12.915$ m at the top and $b_b = 20.574$ m at the bottom and its clear height is $h_o = 3.555$ m. In plan, the bridge is situated perpendicularly to the current of the river connecting two lakes.

The basic dimensions of the bridge are shown in Fig. 1 while the arrangement of the strain gauges (extensometers), dial, and induction gauges in the three cross-sections along the length of the shell is shown in Fig. 2. Views of the bridge from some sides, with the different positions of the ballasting truck on the bridge for three static load schemes in accordance with the test program, are shown in Figs. 3–4 [3].

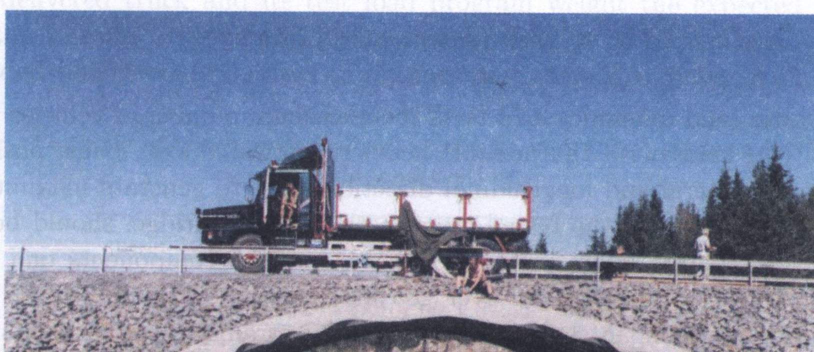

FIG. 3. Side view of bridge and *Scania* ballasting truck during tests (asymmetric load scheme I). Visible tripods for fixing displacement gauges.

FIG. 4. Rear view of ballasting truck on bridge during tests (symmetric load scheme II).

3. Range of static load tests

For static (and dynamic) load tests the maximum number of vehicles (which the span can hold) with the possible heaviest allowable axle loads, positioned in the span's lateral and longitudinal direction so as to generate maximum deflections and strains in the tested cross-sections of the bridge, should be used. In order to generate effort in the selected structural components one *Scania 500143H* truck (total weight of over 285 kN) positioned symmetrically in the cross-section of the span so as to obtain similar strains in the elements of the corrugated plates was used. The vehicle was also symmetrically positioned on the road so as to obtain varied effort in elements of the corrugated plates. It was determined what the proportion of the manifested effects produced by the known test load to those produced by the standard live load according to the current Polish load class B (PN-85/S-10030), i.e., a vehicle at total weight of 255 kN. The ratio of span bending moments under the test load to bending moments under the live moving loads was about 70–75%. The magnitude of the load was considered to be reliable in light of the relevant Swedish (Vågverket – VU 94, Vågverket – 1990:11, Vågverket – 1994:15) and Polish (PN-85/S-10030, PN-89/S-10050) bridge codes.

In the course of the main tests not only the type of ballasting vehicle but also its axle loads were changed from the ones specified in the test program [1], because load of the vehicle considerably exceeded its maximum authorized overload. The weighed front and rear axle loads together considerably exceeded the total weight of the truck (with its load included) of 285 kN. The technical specifications of the ballasting truck were as follows: the inter bumper length – 7.130 m, the width of the body – 2.500 m, the front wheel track of 1.750 m, the front axle to first rear axle spacing – 4.62 m, the spacing between the rear axles – 1.30 m, the maximum weight of the truck with the load – 285.00 kN and without the load – 100.00 kN (the load capacity – 185.00 kN), the front axle load – 82.50 kN, and the weight per rear axles – 2×101.00 kN. The ballasting truck was loaded with sand and weighed immediately before the tests: the particular axles were weighed in turn and then the whole vehicle. The differences in the loads were distributed proportionally to the particular axle loads given in the load specifications of the truck, as shown in Table 1, where the weighed axle loads are given in the top rows while the catalogue axle loads assumed in the test program are given in the bottom rows [1]. The differences in the axle's loads were quite large (over 10%). Considering that during the tests the rear axles (with heavier loads) were located in the critical sections of the span, the weighed axle loads were used in the preliminary calculations to ensure a proper safety margin for the structure. Because of the large differences between the weight of the delivered truck and its test load program weight the expected statical quantities (normal stresses and displacements) had to be recalculated, but the overloaded vehicle could be used to produce much stronger dynamic effects [2].

Table 1. The axle of the *Scania* ballasting trucks and wheel loads in [kN] during static load testing.

Total weight	Axle loads		Wheel loads	
	rear	front	rear	front
285.00	202 = 2×101.00	82.50	50.50	41.25
255.00	178 = 2×89.00	70.70	44.50	35.35

The tests were to be carried out in the full range of static loads and they were to include measurements of vertical (and horizontal) displacements and strains at selected points of the steel shell structure in three cross-sections along the length of the span (Fig. 2): in the middle of the effective span of the shell (in the crown – cross-section I–I), at the end of the reinforcement (cross-section II–II), and in the haunch (cross-section III–III). Since in this kind of structure possible settlement of the continuous foundations had already taken place and considering the fact that bedrock was beneath and no deviations or irregularities in the behaviour of the foundations or work had been observed prior to the bridge tests, continuous foundation settlement was not continuously recorded. Instead it was checked by means of a precision level of the surveyor.

As originally planned, three load schemes (Figs. 3–4), i.e., two asymmetric load schemes (the truck positioned at the protective barrier on the upstream side – scheme I or on the tailwater side – scheme III) and one symmetric (to the longitudinal axis of the bridge) load scheme (the truck positioned on the longitudinal axis of the roadway in such a way that its rear axle was located in the middle of the effective span length – scheme II were used).

Two measuring systems were used: one measuring strains and the other measuring horizontal and vertical displacements (deflections). Each system consisted of three basic components: measurements, control measurements and recording of results.

The influence of changes in atmospheric conditions (mainly temperature) was eliminated through the use of compensation strain gauges in all the measuring points. Taking into account the changeable weather conditions prevailing in northern Sweden at the beginning of April and the proximity of two lakes (high humidity), special quick-drying glue based on synthetic resin was used and the stuck on gauges were coated with a protective weather- and mechanical damage-resistant compound. The stuck on gauges were connected to compensation gauges (mounted on steel plates put against the structure next to the active strain gauges) to form half-bridge circuits.

Before the tests the measuring circuits in the recording, instruments had been calibrated for a fixed displacement value, e.g., 30 mm. The first indications (zero readings) had been obtained before the load was brought onto the span. After the ballasting load was brought onto the span further readings were taken from all the instruments every 10 min for at least 30 min and after unloading until the readings stabilized. If the difference between two consecutive readings was larger than 2%, the load had to remain on the span until the difference was below 2% (PN-85/S-10030, PN-89/S-10050 and PN-77/S-10040). Similarly, readings were taken after unloading the span, i.e., every 10 min for 20 min. At least three such readings were taken. The differences between the last indications of the dial (or inductive) gauges and the electric resistance wire strain gauges after unloading and the initial readings represented the permanent deflections (or strains) and the differences between the total deflections (or strains) and the permanent ones constituted the elastic deflections (or strains).

4. Assessment of displacements and strains measurement accuracy

Probable measuring error δ_f for displacements in the selected points and cross-sections of the corrugated steel plate shell structure for the worst measuring setup was calculated from the following formula (2.2):

$$\delta_{\mathrm{f}} = \sqrt{\delta_1^2 + \delta_2^2 + \delta_3^2 + \delta_4^2} = \sqrt{0.02^2 + 0.01^2 + 0.02^2 + 0.005^2} = \pm 3.04\%, \tag{4.1}$$

where:

$\delta_1 = 2.0\%$ – a displacement converter error;
$\delta_2 = 1.0\%$ – a channel selector compensating unit error;
$\delta_3 = 2.0\%$ – an instrumentation amplifier (bridge) error; and
$\delta_4 = 0.5\%$ – a calibration error.

The probable measuring error for strains δ_ε in the steel shell structure for the best equipment setup was calculated from the following formula (2.5):

$$\delta_\varepsilon = \sqrt{\delta_5^2 + \delta_6^2 + \delta_7^2 + \delta_8^2} = \sqrt{0.02^2 + 0.01^2 + 0.025^2 + 0.005^2} = \pm 3.60\%, \tag{4.2}$$

where:

$\delta_5 = 2.0\%$ – a strain gauge error;
$\delta_6 = 1.0\%$ – a channel selector compensating unit error;
$\delta_7 = 2.5\%$ – an instrumentation amplifier error; and
$\delta_8 = 0.5\%$ – a steel elasticity modulus error.

5. Results of numerical calculations

As part of the test program (also carried out by the Canadian firm *AIL*) the ordinates of the influence lines of the bending moments and deflections (vertical displacements) in the particular cross-sections of the shell structure made from corrugated plates were computed (using software *SODA, ver 3.5*) for the three ballasting truck positions and the actual strength parameters of the steel and the backfill. 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.

The influence lines of the transverse distribution of the load among the particular corrugated plates were used in the computations of expected deflections f, strains ε, and normal stresses σ. 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 computations were performed for the actual positions and axle loads of the ballasting vehicles. The ordinates of the influence lines under the axles were read directly from the computer printouts to avoid the unnecessary, highly laborious, and less accurate interpolation of the ordinates. The calculations done by *AIL* 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 program *FLAC* from the nonlinear contact elements of interface type.

It follows from the above that other numerical methods should be sought and assumptions that are more realistic should be made when building computational models of such flexible structures. Currently the writers of this paper are in possession of an excellent computing program called *Fast Lagrangian Analysis of Continua FLAC, ver 3.5* (courtesy of the Arcadia Ekokonrem company of Wrocław) which makes possible a comprehensive and accurate analysis of such complex steel-soil structures and allows one to model the interface between the different materials. Preliminary analyses and computations performed by means of this program yielded values much closer to the test results. This will be reported in detail in other paper [3]. The new finite element procedures used for the soil-structures interaction 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 finite element 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 of the culvert as infinitely long.

The analyses are performed gradually, beginning with the structure resting on its foundation with backfill. The placement of the first layer of backfill alongside the culvert is modelled by adding the first layer of soil elements to the finite element mesh. At the same time, loads representing the weights of the added elements are applied. 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.

The soil is modelled as elastic-plastic model (criterion Coulomb-Mohr), with linear modulus variations with depth. Modulus variation $E(z) = E_o + mz$ is defined using surface modulus E_o and modulus gradient m. Parameters used to model for 95% Standard Proctor are following: Poisson's ratio $v = 0.17$; cohesion $c = 0$, friction angle $\phi = 43°$; gradient $m = 3.8$ MPa/m; $E_o = 20$ MPa and unit weight of soil $\gamma = 20$ kN/m^3. The steel structure of box culvert type was modelled as bilinear elastic with material constants of: initial Young's modulus $E_1 = 207$ GPa; secondary Young's modulus $E_2 = 12$ GPa; Poisson's ratio $v = 0.30$; yield stress $\sigma_y = 282$ MPa. The asphalt material was considered linear elastic with Young's modulus $E = 6.9$ GPa and Poisson's ratio $v = 0.41$.

The original formulation of MOORE and BRACHMAN [15] has been modified to incorporate orthotropic shell elements, which are based on the harmonic axisymmetric shell elements of ROTTER and JUMIKIS [16] but have been modified in two 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.

The behaviour of the flexible structure of box culvert type is dependent on a large degree on their interaction with surrounding 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 freestanding structure. It is this aspect of their behaviour that makes the use of soil-structure interaction analyses, with simulation of behaviour of both backfill and shell structure, absolutely essential to provide a realistic basis for design.

Some graphs of the calculated displacements and unit strains in the particular points, elements and cross-sections of the steel shell structure are shown in Figs. 6, 8 and 9 (red colour).

6. RESULTS OF MEASUREMENT

6.1. *Vertical displacements*

Courses of vertical displacements in the selected points of the steel shell structure are shown in Fig. 5 while the diagrams of maximum vertical displacements in the transverse direction of the shell for the three load schemes are shown in Fig. 6.

6.2. *Horizontal displacements*

The day before the main tests were to be carried out, i.e., on August 18, 2002, measurements of vertical and horizontal displacements under an unknown load (moving trucks weighing nearly the same or more than the vehicle used for ballasting the bridge on August 19, 2002) were performed. Since the preliminarily measured horizontal displacements were slight (within measuring or reading errors), a decision was made to measure only vertical displacements during the main tests.

6.3. *Strains*

Some graphs of unit strains in the particular points, elements, and cross-sections of the steel shell structure are shown in Fig. 7 while the diagrams of maximum strains across the span in the selected points in the three tested cross-sections, i.e., in the crown, at the end of the crown's reinforcement, and in the haunch of the shell, for the three static load schemes are shown in Figs. 8 and 9.

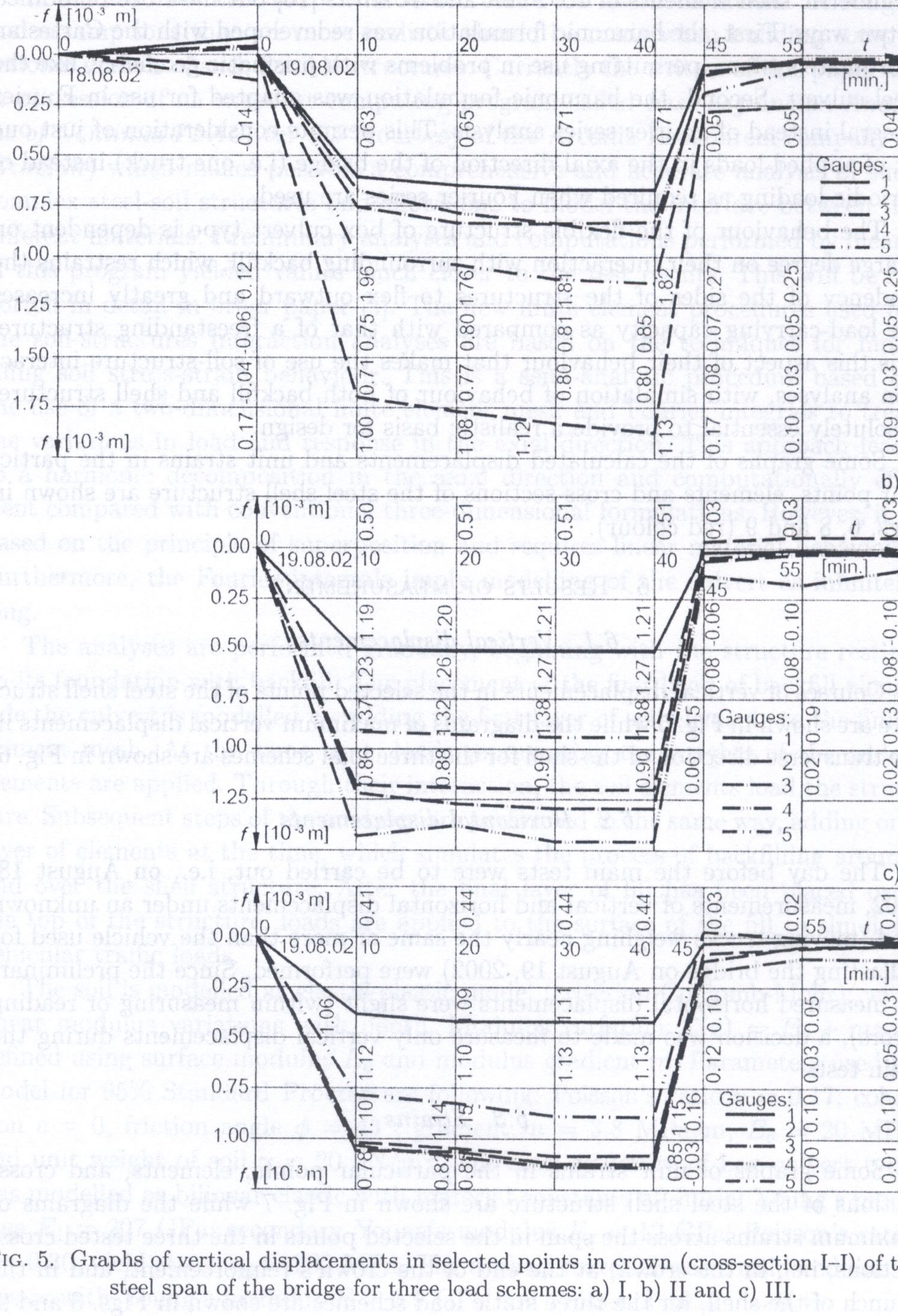

FIG. 5. Graphs of vertical displacements in selected points in crown (cross-section I–I) of the steel span of the bridge for three load schemes: a) I, b) II and c) III.

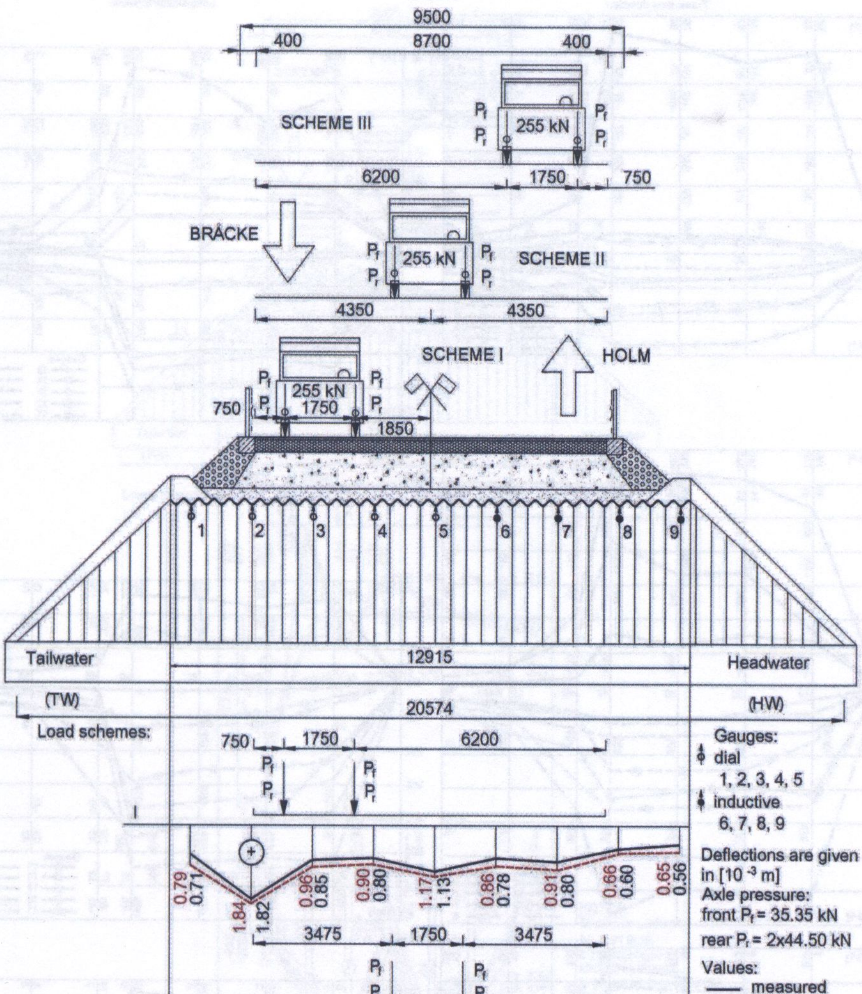

FIG. 6. Graphs of maximum vertical displacements in transverse direction of the span for three test load schemes.

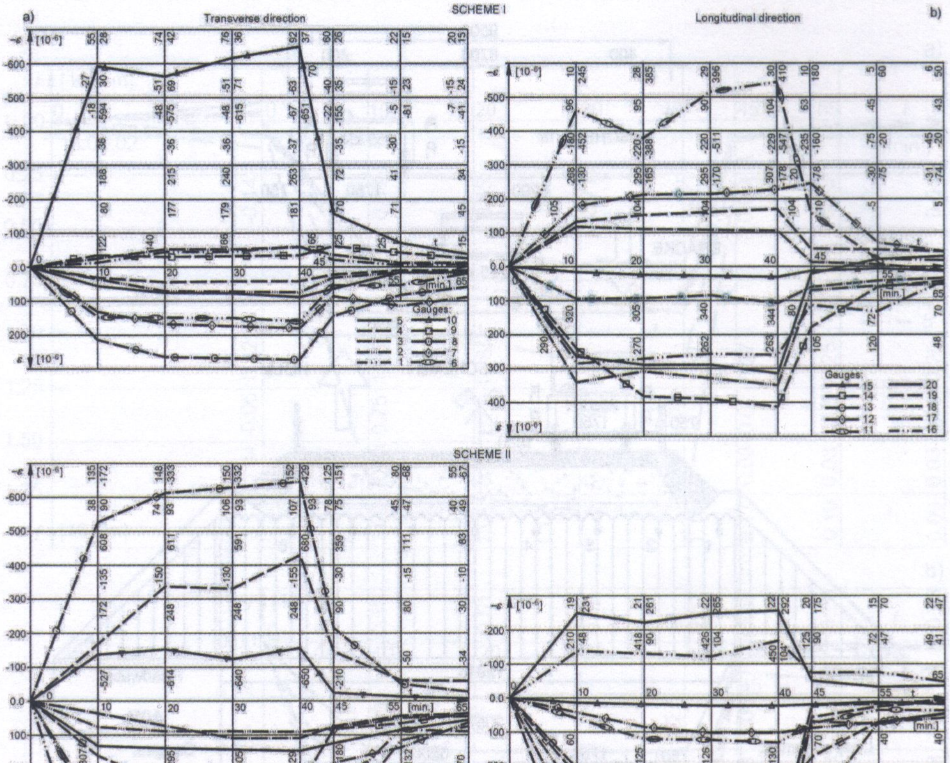

FIG. 7. Graphs of strains in selected points of steel shell in cross-section II–II in: a) transverse direction – gauges no. 1–10 and b) longitudinal direction – gauges no. 11–20 for load schemes I, II and III.

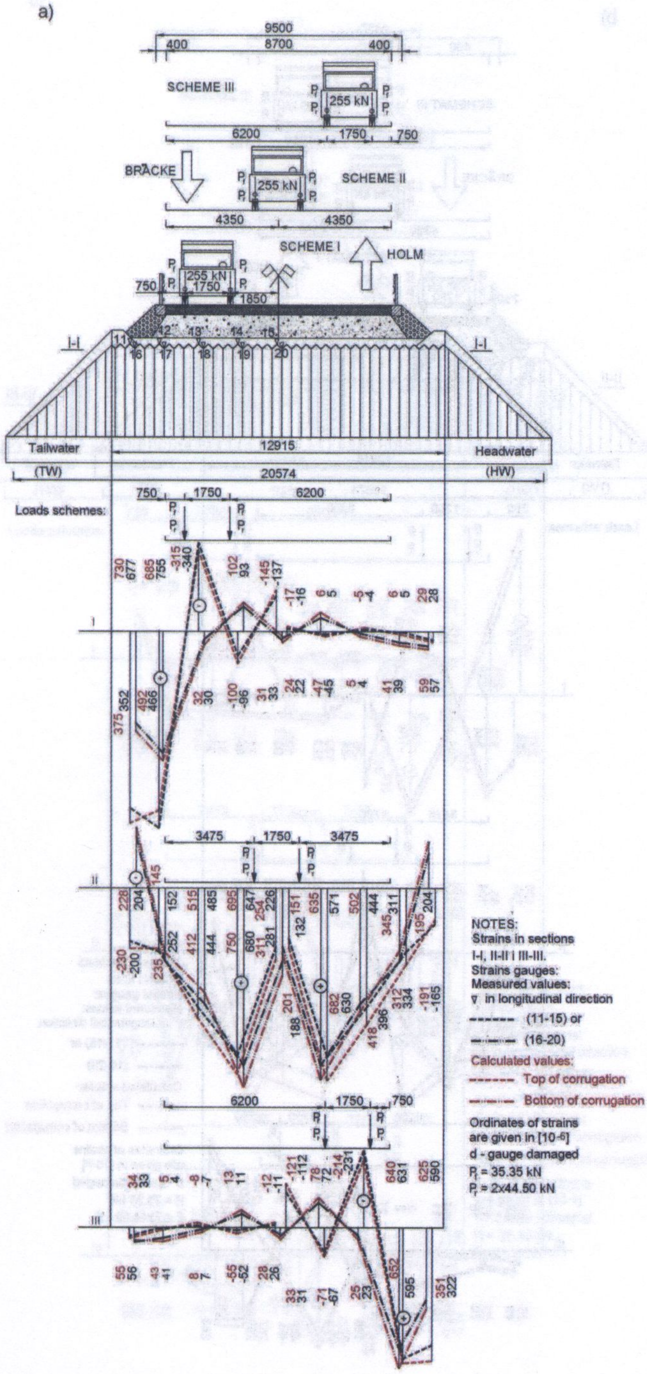

FIG. 8a. Graphs of maximum strains in selected points (gauges no. 1–20 in transverse direction) of steel shell structure in cross-sections of the span: a) I–I.

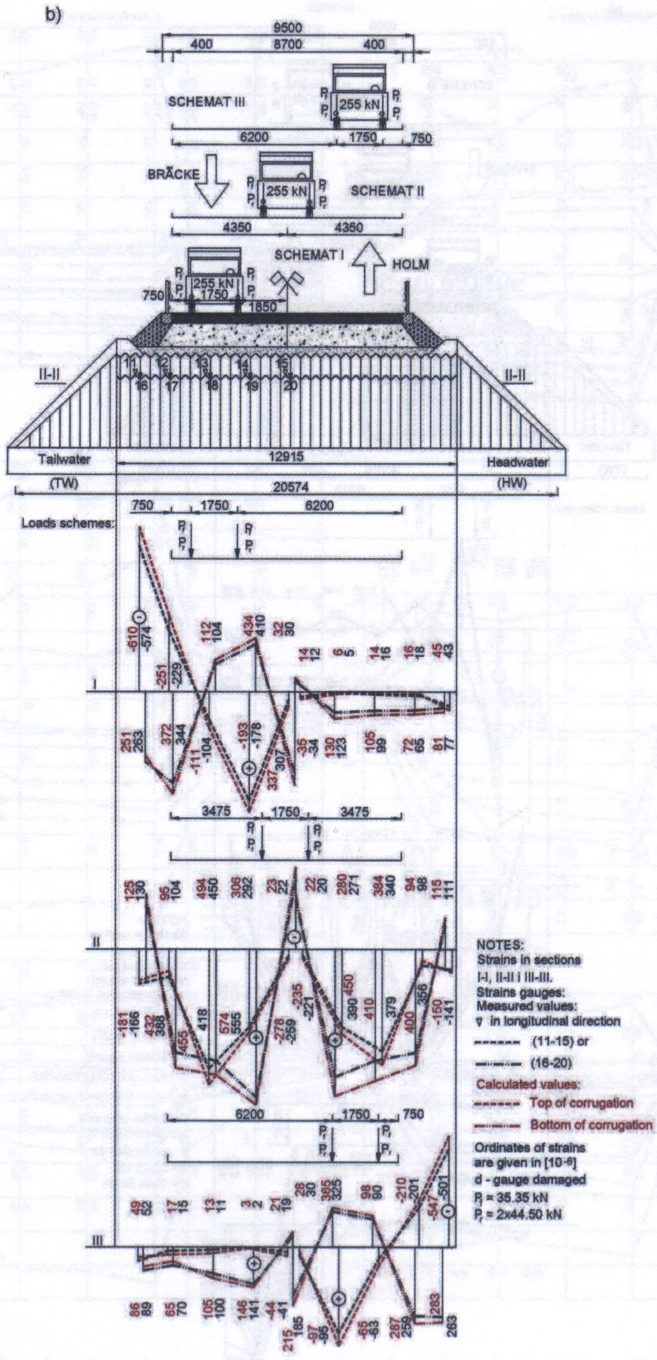

FIG. 8b. Graphs of maximum strains in selected points (gauges no. 1–20 in transverse direction) of steel shell structure in cross-sections of the span: b) II–II.

c)

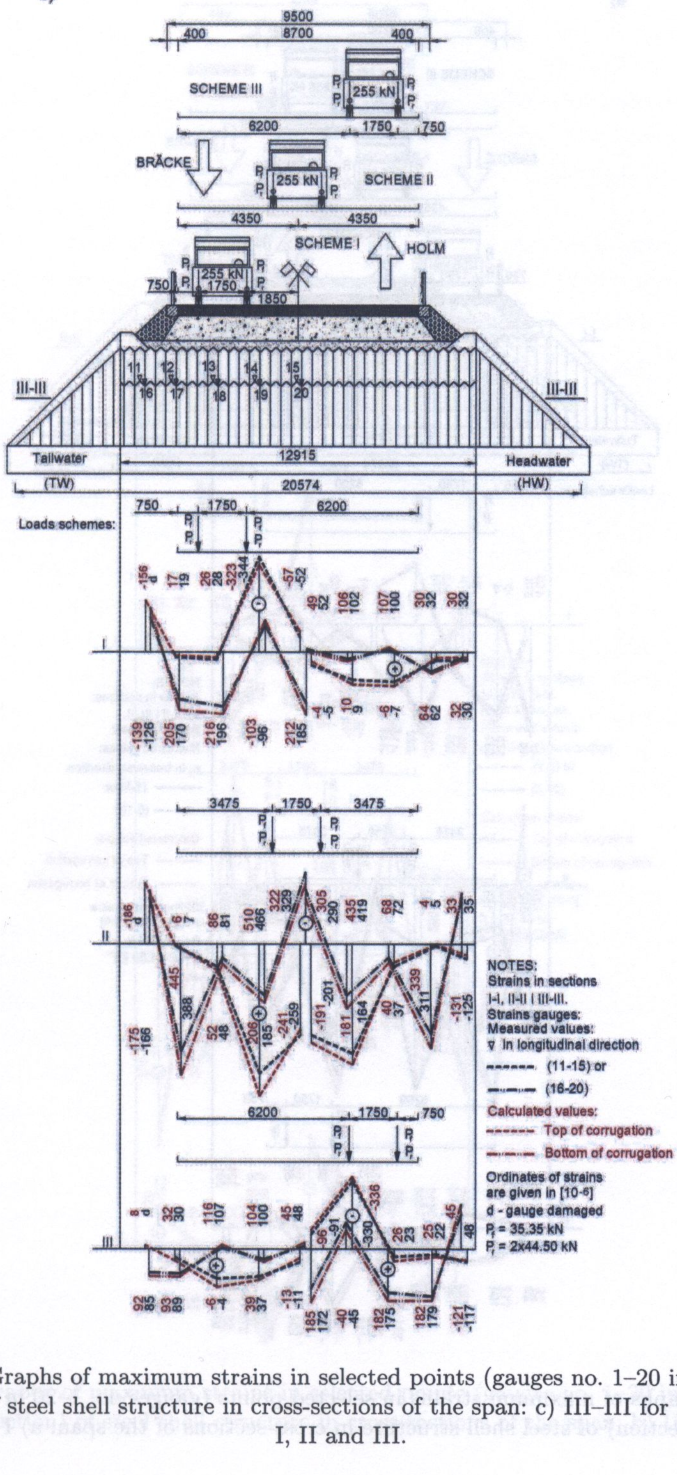

FIG. 8c. Graphs of maximum strains in selected points (gauges no. 1–20 in transverse direction) of steel shell structure in cross-sections of the span: c) III–III for load schemes I, II and III.

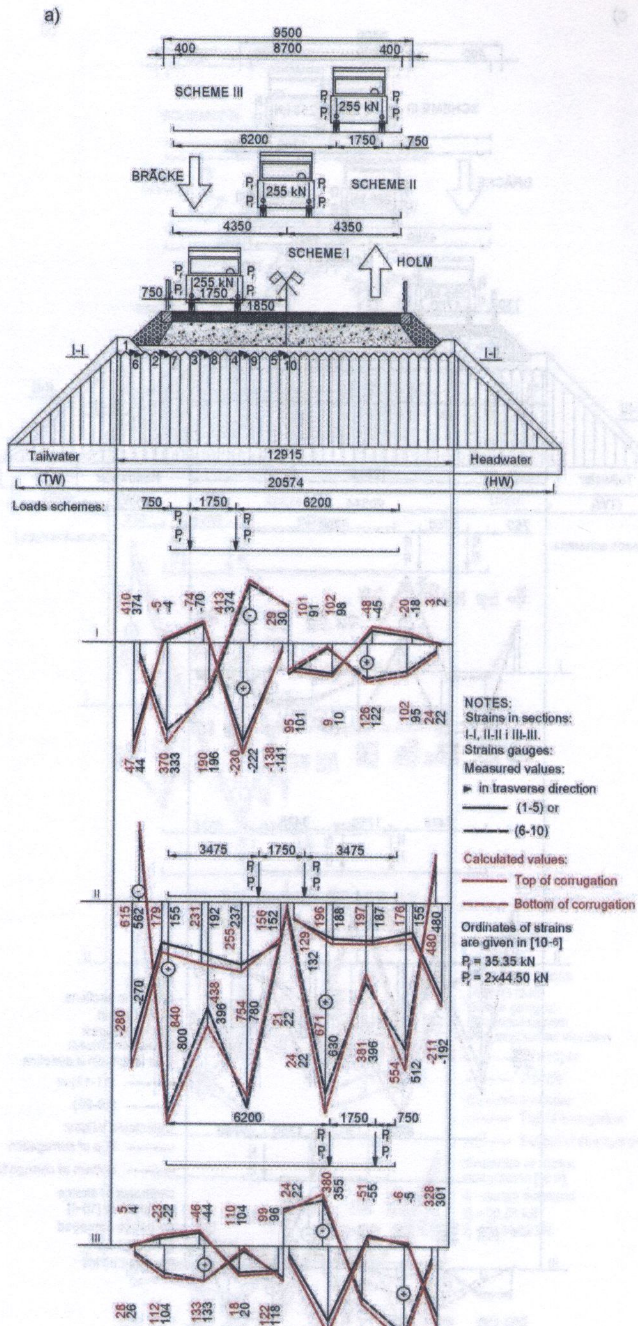

FIG. 9a. Graphs of maximum strains in selected points (gauges no. 1–20 in longitudinal direction) of steel shell structure in cross-sections of the span: a) I–I.

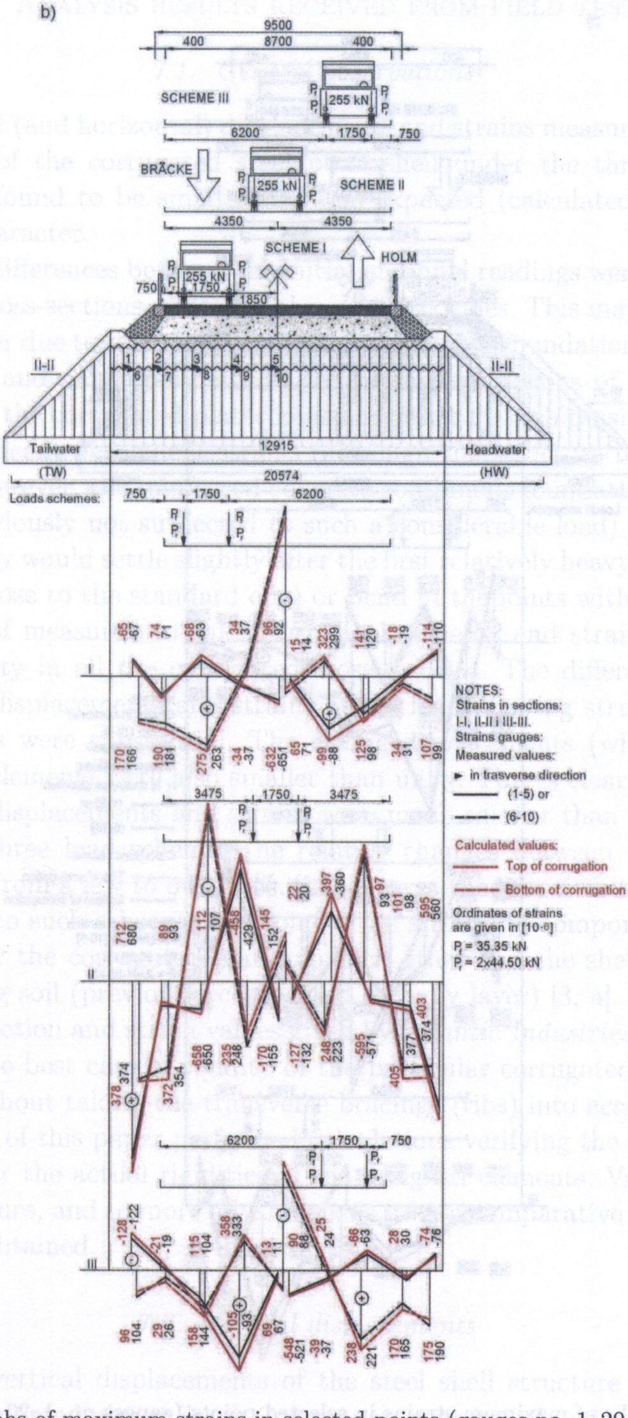

FIG. 9b. Graphs of maximum strains in selected points (gauges no. 1–20 in longitudinal direction) of steel shell structure in cross-sections of the span: b) II–II.

c)

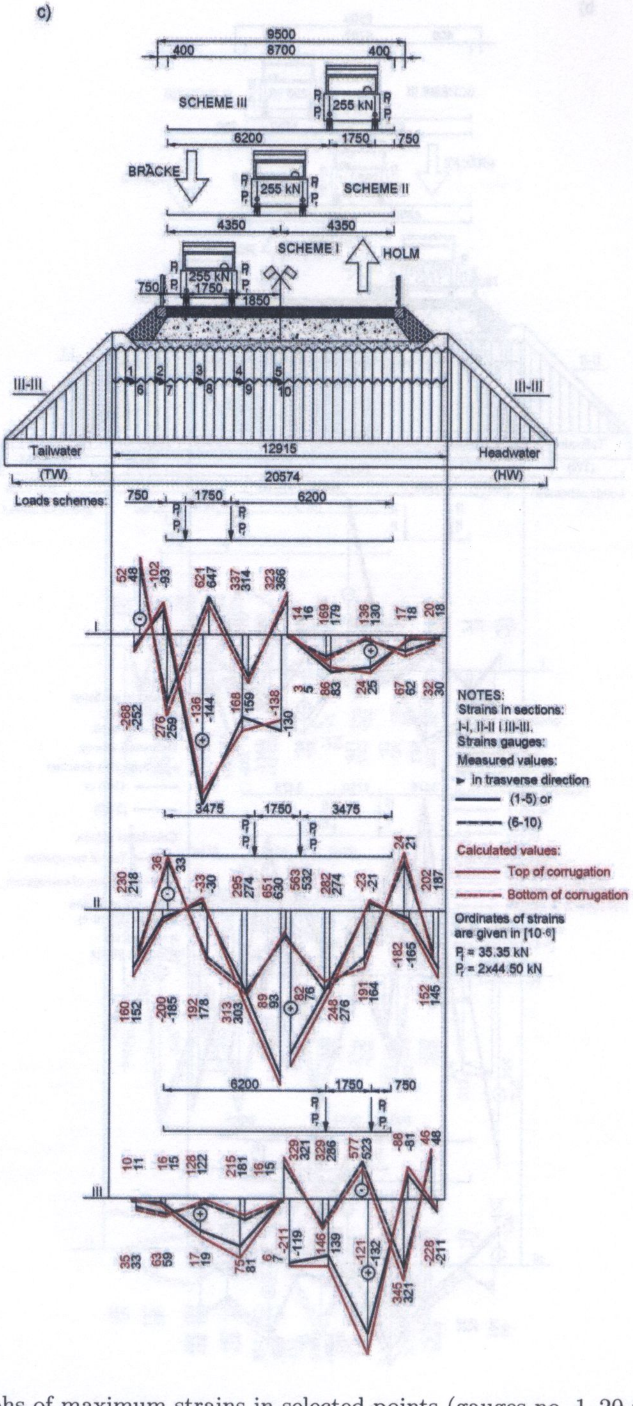

FIG. 9c. Graphs of maximum strains in selected points (gauges no. 1–20 in longitudinal direction) of steel shell structure in cross-sections of the span: c) III–III.

7. Analysis results received from field tests

7.1. *General observations*

The vertical (and horizontal) displacements and strains measured in the three cross-sections of the corrugated steel plate shell under the three static load schemes were found to be smaller that the expected (calculated) ones and to have elastic character.

The slight differences between the initial and final readings were not identical in the tested cross-sections under the three load schemes. This may indicate that they were rather due to the settlement of the continuous foundations and possible reading errors and only minimally to the permanent strains of the steel shell structure since the corrugated plates joined together by high tensile bolts should not show significant permanent strains (although the structure took quite long to distress). However, the reinforced concrete continuous foundations (supports) were new (previously not subjected to such a considerable load) and one could expect that they would settle slightly after the first relatively heavy (over 250 kN) service load (close to the standard one) or bend in the joints with the supports.

The ratio of measured to calculated displacements and strains was always lower than unity in all the considered cross-sections. The differences between the measured displacements and strains of the load-carrying structure and the calculated ones were substantial. The elastic displacements (with or without foundation settlement) were also smaller than unity. This is clear evidence that the measured displacements and strains were much smaller than the calculated ones. For all three load schemes the relative changes between the respective results ranged from a few to over 18%. Since it was the first time that the bridge was subjected to such a heavy load some of its structural components adjusted to one another: the corrugated plates to each other and the shell structure to the surrounding soil (previously compacted layer by layer) [3, 4]. Moreover, the calculated deflection and strain values given by *Atlantic Industries Limited* were obtained for the best case of rigidity of the particular corrugated plates in the bridge, i.e., without taking the transverse bracings (ribs) into account.

The writers of this paper performed calculations verifying the deflection and strain values for the actual rigidities of the designed elements. Values closer to the measured ones, and so more advantageous from a comparative analysis point of view, were obtained.

7.2. *Vertical displacements*

Maximum vertical displacements of the steel shell structure of the bridge made from corrugated plates *Super Cor* type were obtained under load scheme I (Fig. 6 and Table 2), i.e., when the ballasting truck was asymmetrically (if one

Table 2. Maximum vertical displacement values in $[10^{-3}\ \mathrm{m}]$ in selected points of steel shell bridge obtained from measures and calculations during field tests according to static loads

No. gauges	Load schemes: I				II				III			
	f_m	f_c	$(f_m - f_c)/f_m$ [%]	f_m/f_c	f_m	f_c	$(f_m - f_c)/f_m$ [%]	f_m/f_c	f_m	f_c	$(f_m - f_c)/f_m$ [%]	f_m/f_c
1	0.71	0.79	–11.26	0.90	0.51	0.48	5.88	1.06	0.44	0.46	–4.54	0.95
2	1.82	1.84	–1.09	0.99	1.21	1.17	3.30	1.04	1.11	1.13	–1.80	0.98
3	0.85	0.96	–12.94	0.88	1.27	1.30	–2.36	0.97	1.13	1.19	–5.31	0.95
4	0.80	0.90	–12.50	0.88	1.38	1.36	1.44	1.01	1.15	1.20	–4.35	0.96
5	1.13	1.17	–3.54	0.96	0.90	1.02	–13.33	0.88	0.85	0.96	–12.94	0.88
6	0.78	0.86	–10.25	0.91	0.95	1.04	–9.47	0.91	1.67	1.70	–1.79	0.98
7	0.80	0.91	–13.75	0.88	0.95	1.06	–11.57	0.89	1.21	1.35	–11.57	0.89
8	0.60	0.66	–10.00	0.91	0.82	0.96	–17.07	0.85	1.32	1.30	1.51	1.01
9	0.56	0.65	–16.07	0.86	0.50	0.48	4.00	1.04	0.50	0.59	–18.00	0.85

Notes: Measured f_m and calculated f_c vertical displacements.

faces in the transverse direction of the span from the tailwater side) positioned near the protective barrier in the middle of the effective span (in the longitudinal direction). The displacements amounted to 1.82×10^{-3} m and occurred under the outer wheel of the truck on the tailwater side. Under (symmetrical) load scheme II, the largest displacements of the load-carrying structure amounted to 1.38×10^{-3} m and occurred directly under the wheels of the ballasting vehicle. Under load scheme III – the vehicle positioned asymmetrically, but conversely relative to scheme I, in the middle of the effective span (in the crown) – the maximum displacements amounted to merely 1.15×10^{-3} m and they occurred on that half of the roadway of the bridge where the gauges were located.

It should be noted here that since the shell structure of the bridge is symmetrical, both in the longitudinal and transverse direction, the dial gauges and the induction gauges measuring vertical displacements were placed only up to half of the width of the span. Thus the vertical displacements registered under load scheme III are not maximum and they are incomparable with the other two schemes since the ballasting truck, and so its axles loads, were located on the span's side (roadway) opposite to the side on which the dial gauges and the induction (displacement) gauges were installed and therefore they affected the other side of the bridge only to a slight degree, which explains why lower displacement values were obtained on that side of the span. Thus one can assume that the maximum vertical displacements registered under load scheme I are also the maximum values for load scheme III and the displacements registered under load scheme III are directly applicable to the other side of the span subjected to the scheme load I.

7.3. Strains

The unit strains measured in load-carrying elements of the corrugated plates (shown in Fig. 7 and in Tables 3 and 4 where the values are given at the top and bottom of the corrugations) were mostly smaller than the calculated strains (normal stresses) in the three analyzed cross-sections along the height of the structure, except for a few points (Fig. 2) along the length of the bridge's span subjected to quite large concentrated forces under the three static load schemes.

The maximum unit strains (normal stresses) in the steel shell structure registered under the static load occurred in cross-section I–I, i.e., in the crown of the structure, and they were as follows:

a) under load scheme I in the transverse direction of the span (Fig. 8a) they occurred in the top edges of the corrugations directly under the outer wheel of the vehicle and amounted to $\varepsilon_{ym} = 755 \times 10^{-6}$ ($\sigma_{ym} = 155.00$ MPa) while in the longitudinal direction (Fig. 9a) they also occurred in the top edges of the corrugation directly under the wheels of the ballasting vehicle and amounted to $\varepsilon_{xm} = 374 \times 10^{-6}$ ($\sigma_{xm} = 76.50$ MPa);

b) under load scheme II in the longitudinal direction (Fig. 9a) of the shell structure they occurred in the bottom edges of the corrugations under the wheels of the ballasting vehicle and they amounted to $\varepsilon_{xm} = 800 \times 10^{-6}$ ($\sigma_{xm} = 164.00$ MPa) while in the transverse direction of the span (Fig. 8a) they also occurred in the bottom edges of the corrugations and amounted to $\varepsilon_{ym} = 680 \times 10^{-6}$ ($\sigma_{ym} = 139.50$ MPa); and

c) under load scheme III in the transverse direction of the span (Fig. 8a) they occurred in the top edges of the corrugations and amounted to merely $\varepsilon_{ym} = 631 \times 10^{-6}$ ($\sigma_{ym} = 129.50$ MPa) while in the longitudinal direction of the span (Fig. 9a) they also occurred in the bottom edges of the corrugations and amounted to $\varepsilon_{xm} = 376 \times 10^{-6}$ ($\sigma_{xm} = 77.00$ MPa), but in this case the ballasting load was positioned on the side opposite to the side where the strain gauges were installed.

For cross-section II–II (at the end of the crown's reinforcement) of the steel shell structure of the bridge the following maximum unit strains and calculated on their basis normal stresses were obtained:

a) under load scheme I in the longitudinal direction of the shell structure (Fig. 9b) they occurred in the bottom edges of the corrugations and amounted to $\varepsilon_{xm} = -651 \times 10^{-6}$ ($\sigma_{xm} = -133.50$ MPa) while in the transverse direction of the span (Fig. 8b) they occurred in the top edges of the corrugations, were directed towards the end of the span, and amounted to $\varepsilon_{ym} = -574 \times 10^{-6}$ ($\sigma_{ym} = -117.50$ MPa);

b) under load scheme II in the longitudinal direction of the shell (Fig. 9b) they occurred in the top edges of the corrugations towards the end of the bridge and amounted to $\varepsilon_{xm} = 680 \times 10^{-6}$ ($\sigma_{xm} = 139.50$ MPa) while the maximum strains (normal stresses) in the transverse direction of the span (Fig. 8b) occurred under the wheels of the ballasting truck and amounted to $\varepsilon_{ym} = 555 \times 10^{-6}$ ($\sigma_{ym} = 114.00$ MPa); and

c) under load scheme III in the transverse direction of the shell structure (Fig. 8b) they occurred in the top edges of the corrugations in the middle of the width of the span and amounted to $\varepsilon_{ym} = -501 \times 10^{-6}$ ($\sigma_{ym} = -103.00$ MPa) while in the longitudinal direction of the span (Fig. 9b) they also occurred in the bottom edges of the corrugations and amounted to merely $\varepsilon_{xm} = -521 \times 10^{-6}$ ($\sigma_{xm} = -107.00$ MPa).

In the last analyzed cross-section (III–III in the haunch of the shell) the maximum unit strains and calculated on their basis normal stresses under the three load schemes were as follows:

a) under load scheme I in the longitudinal direction of the span (Fig. 9c) they occurred in the top edges of the corrugations directly under the wheels of the ballasting truck and amounted to $\varepsilon_{xm} = 647 \times 10^{-6}$ ($\sigma_{xm} = 132.50$ MPa) while in the transverse direction of the span (Fig. 8c) they also occurred in the top edges of the corrugations and amounted to $\varepsilon_{ym} = -344 \times 10^{-6}$ ($\sigma_{ym} = -70.50$ MPa);
b) under load scheme II in the longitudinal direction of the shell (Fig. 9c) they occurred in the top edges of the corrugations between the wheels of the ballasting truck and amounted to $\varepsilon_{xm} = 630 \times 10^{-6}$ ($\sigma_{xm} = 129.00$ MPa) while in the transverse direction of the span (Fig. 8c) they were also located in the top edges of the corrugation between the wheels of the truck and amounted to $\varepsilon_{ym} = 466 \times 10^{-6}$ ($\sigma_{ym} = 95.50$ MPa); and
c) under load scheme III in the longitudinal direction of the shell (Fig. 9c) they occurred in the top edges of the corrugations in the middle of width of the bridge roadway and amounted to $\varepsilon_{xm} = 523 \times 10^{-6}$ ($\sigma_{xm} = 107.00$ MPa) while in the transverse direction of the span (Fig. 8c) they also occurred in the top edges of the corrugations and amounted to merely $\varepsilon_{ym} = -330 \times 10^{-6}$ ($\sigma_{ym} = -67.50$ MPa).

The unit strains (normal stresses), similarly as the deflections (vertical displacements), of the steel shell structure during the static tests and construction generally returned to their original position [1], [2]. The behaviour of the shell structure did not raise any suspicions. Only the rather slow return of the gauges' indications to zero, particularly as the backfill layers were being laid, and the rather long time the backfill ground took to stabilize might raise some suspicions, but based on previous tests carried on such structures this behaviour could be regarded as quite normal.

Similar conclusions were also drawn from an analysis of the linear distributions of normal stresses in a few cross-sections of the steel shell of the bridge and from an analysis of the interaction between the individual corrugated plates and between the steel shell structure and the surrounding backfill, made on the basis of diagrams of the strains and displacements of the steel shell structure.

A comparison of the results shows that the expected strain (normal stress) values were much higher than the ones determined from the measured strains. This indicates, similarly to the deflections (horizontal and vertical displacements) of the steel shell of the bridge and previous tests carried out on structures of this type, considerable load-capacity reserves.

Table 3. Maximum measured and calculated strain values in $[10^{-6}]$ in longitudinal direction in selected three 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}	ε_{xc}	$(\varepsilon_{xm}-\varepsilon_{xc})/\varepsilon_{xm}$ [%]	$\varepsilon_{xm}/\varepsilon_{xc}$	ε_{xm}	ε_{xc}	$(\varepsilon_{xm}/\varepsilon_{xc})/\varepsilon_{xm}$ [%]	$\varepsilon_{xm}/\varepsilon_{xc}$	ε_{xm}	ε_{xc}	$(\varepsilon_{xm}-\varepsilon_{xc})/\varepsilon_{xm}$ [%]	$\varepsilon_{xm}/\varepsilon_{xc}$
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: strains in longitudinal direction at top and bottom of corrugations respectively: measured ε_{xm} and calculated ε_{xc}.

Table 4. Maximum measured and calculated strain values in $[10^{-6}]$ at transverse direction in selected three 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_{ym}	ε_{yc}	$(\varepsilon_{ym}-\varepsilon_{yc})/\varepsilon_{ym}$ [%]	$\varepsilon_{ym}/\varepsilon_{yc}$	ε_{ym}	ε_{yc}	$(\varepsilon_{ym}/\varepsilon_{yc})/\varepsilon_{ym}$ [%]	$\varepsilon_{ym}/\varepsilon_{yc}$	ε_{ym}	ε_{yc}	$(\varepsilon_{ym}-\varepsilon_{yc})/\varepsilon_{ym}$ [%]	$\varepsilon_{ym}/\varepsilon_{yc}$
at bottom edges												
I	466	492	–5.58	0.95	344	372	–8.14	0.92	196	216	–10.20	0.91
II	680	750	–10.29	0.91	555	574	–3.42	0.96	388	445	–14.69	0.87
III	592	652	–10.13	0.91	259	287	–10.81	0.90	179	182	–1.67	0.98
at top edges												
I	755	685	–9.27	1.10	–574	–610	6.27	0.94	–344	–323	6.10	1.06
II	647	696	–7.57	0.93	450	494	–9.77	0.91	466	510	–9.44	0.91
III	631	640	–1.43	0.98	–501	–547	9.18	0.92	–330	–336	1.82	0.98

Notes: strains in transverse direction at top and bottom of corrugations respectively: measured ε_{ym} and calculated ε_{yc}.

7.4. *Standard bridge acceptance conditions*

In order for the results from the tests performed in the course of construction of the Gimån bridge, including its trial loading to be considered as satisfactory according to the Polish standards for typical steel bridges (PN-77/S-10040, PN-82/S-10052, PN-85/S-10030 and PN-89/S-10050), the following conditions must be fulfilled:

a) the calculated and measured displacement values must be similar;
b) there must be no damage to the structural components or their joints occurred as a result of the performed tests;
c) the steel shell structure must deform elastically in the range of allowable displacements under loads close to the standard load and the displacements cannot exceed the calculated values; and
d) the permanent deflections (displacements) cannot amount to more than 25% of the elastic deflections.

It should be noted here that guidelines or regulations for structures of this type do not yet exist in Poland whereby the contractors often have problems with the acceptance of such bridges by the bridge administration. Preliminary guidelines for dimensioning such flexible structures (with a maximum span of 10 m) have been developed by Pettersson and Sundquist [17] in Sweden, but they should be modified for the Polish conditions and the proposed algorithms should be improved if they are to be applied to structures with much larger effective spans, e.g., over 25 m.

8. Final conclusions

As a result of the tests, carried out on the road bridge during the construction of the shell and under the main test static load, the vertical displacements and unit strains (indirectly normal stresses) of the load-carrying structure were determined and compared with the calculated values. Based on the practical experience gained from the tests, the observations concerning the behaviour of the shell structure in this type of bridge made during the tests and a comprehensive analysis of the measurement and computation results, the following general conclusions about the actual behaviour of the bridge can be drawn:

1. The performance of the flexible shell made from *Super Cor* corrugated steel plates was beyond reproach. The average values of the measured displacements and strains were lower than the ones computed for the same load. This became even more apparent when the permanent displacements and possible slight settlement of the supports were taken into account. This was clear evidence of a much

greater stiffness of the span than the one assumed in the static-strength calculations in which such a degree of interaction between the steel shell structure and the surrounding backfill was not foreseen. The final conclusions regarding the performance of such flexible bridge structures can be highly useful in developing design, acceptance, and other guidelines.

2. The displacements and strains measured at the top and bottom of the corrugations of the steel shell structure, made from corrugated plates joined together entirely with high tensile bolts, caused by the test static load and by the backfill load during the construction of the bridge [3, 4] had practically elastic character and were smaller than the calculated ones in almost all the considered points and cross-sections of the steel shell structure. The measured permanent displacements and strains differed slightly between the particular cross-sections of the steel shell structure of the tested bridge and they were not proportional to the elastic deflections and strains. The distribution of the elastic deflections and strains of the superstructure in the transverse direction of the span obtained from the measurements was curvilinear, but with much smaller curvatures than in the case of the theoretical distribution, the reason being that the slight permanent displacements registered during the tests were mostly due to the settlement of the supports (continuous foundations), the bending of the plates at the joint with the supports, reading errors or instrumentation measuring errors (changes in temperature and air humidity during the measurements), and only to a slight degree to the permanent strains of the load-carrying structure itself (below 2% of the total deflections).

3. A comparison of the displacements and strains measured in the particular cross-sections of the load-carrying structure with the calculated ones shows considerable differences between them to the advantage of the safety of the steel shell structure. The causes of the differences lie in the calculations in which the stiffness of the cross-section of the span was probably too conservatively estimated, which is to the advantage of the bridge's safety. This is also evidence of much greater rigidity of the cross-section of the shell structure, which can be attributed to better interaction between the steel elements of the steel shell structure and the surrounding backfill than the one assumed in the static calculations and to the more beneficial effect of the quite rigid road surface, resulting in a reduction in the unit pressures of the ballasting vehicle on the shell structure owing to the distribution of the static load over a larger surface and to the contribution of the flexibility of the steel structure to the carrying of loads. Another possible reason may be that too large rear axle loads were assumed in the numerical computations. Sometimes the differences amount to over 18%. Moreover, the displacement and strain (normal stress) values were computed in the nodes of the computational model grid but not all the investigated cross-sections (points) coincided with the nodes. Therefore, the deflections and strains of the

load-carrying structure in these cross-sections were determined through interpolation between the assumed nodes of the grid. Besides, in model grid structures there is much better interaction between the structural components than it was assumed in the initial test program. This is especially true for the transverse distribution of the load among the particular structural components (corrugated plates). With regard to normal stresses, the respective results were found to be in better agreement.

4. As the effect of executed calculations by the FEM 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 two-dimensional $2D$ analysis) with the contact elements of the *interface* type between steel shell and backfill. In the some special cases, the calculations were possible also to execute in the three-dimensional space $3D$ with the aim of more detailed analysis. 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.

5. The registered settlements of the continuous foundations (supports) were slight and they could be ascribed to reading inaccuracies or instrumentation measuring errors or to the irregularities in the contact between the elements of the plates and the foundation rather than to the actual settlement. Since the supports had not been subjected to such a heavy load before, one could expect that they would settle slightly under the first considerable load. The settlements were found to be so slight that their influence on the other deflection values was neglected, especially in view of the considerable load capacity reserves in the shell structure of the bridge.

6. The position of the neutral axis in the cross-sections of the load-carrying structure and the magnitude of the normal stresses indicate that the steel shell structure made from corrugated plates interacts very well with both the surrounding soil and the pavement laid on the backfill, which significantly affects the level of displacements and strains. The neutral axis of the cross-section of the steel structure is located slightly higher than it follows from the strength calculations.

7. The slight differences between the initial and final readings show that the slight permanent displacements of the load-carrying structure of the steel shell were mainly due to possible settlement of the continuous foundations, backfilling, and compaction of the backfill and only to a small extent to the permanent deflections of the structure made from corrugated plates joined together with high tensile bolts.

8. The high tensile bolt joints were found to be in good condition and did not raise any suspicions. The tension forces in the bolts were of the proper magnitude (AISI – Handbook of Steel Drainage & Highway Construction Products, Fifth Edition, Chicago 1994).

9. Close inspection of the bridge after the tests and during supplementary and control measurements showed that the bridge did not reveal any damage to the structural components.

10. Having passed the static load test, the bridge was accepted for dynamic load testing and ultimately for normal service under a load of 400 kN (corresponding to the Polish class B load).

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