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Experimental Verification of Bearing Capacity of Composite Truss Girders

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Summary

A frequently applied roof form covering industrial shops is a structure which consists of thin-walled reinforced concrete precast slabs laid on steel truss girders. The structural continuity is assured by means of reinforced concrete curbs cast-in-place on the truss top flanges. A statical analysis of such a structure considered as a composite one had been performed. This model represents quite well the real work of the structure. The calculation results have been positively veryfied by means of experimental research.

1. Introduction

A blocked, five bay production shop of a copper ore enrichment plant has its dimensions in plan 225.6 x 135.0 m. The cross-section of the production building is shown in Fig. 1, while the general view of the researched truss girders is presented in Fig. 4. The bays have their heights respectively 18, 25, 34, 25 and 18 m. Each bay is fitted out with two overhead cranes. The heaviest overhead crane has its bearing capacity of 160 T. The main load carrying structure consists of steel and reinforced concrete frames spaced every 6.0 m. The necessary rigidity in the plan of frames is provided by the middle bay, constructed in the form of reinforced concrete, monolithic bunkers for crumbled ore. Arrangement of longitudinal and transversal walls of these bunkers can be considered as a solidly reinforced concrete shaft.

Simply-supported steel truss girders of 30, 30, 15, 30 and 30 m span and laid on them prefabricated thin-walled reinforced concrete slabs, of 6 m span, are used as structural elements of the roof. These slabs have been manufactured in Poland for 45 years as typical elements of industrial shop roofs. Partial corrosion of top flanges of roof girders has been observed after 22 years of exploitation. The object should be still exploited for at least 30 years. There exists consequently a necessity to qualify the bearing capacity of roof girders.

Bearing capacity had been qualified by means of: 1) a standard calculation procedure as for flat lattice elements, taking into consideration reduction of top flange cross-section because of corrosion; 2) taking into consideration a composite model of a truss girder top flange. The calculated bearing capacity of truss girders is greater than the acting load.

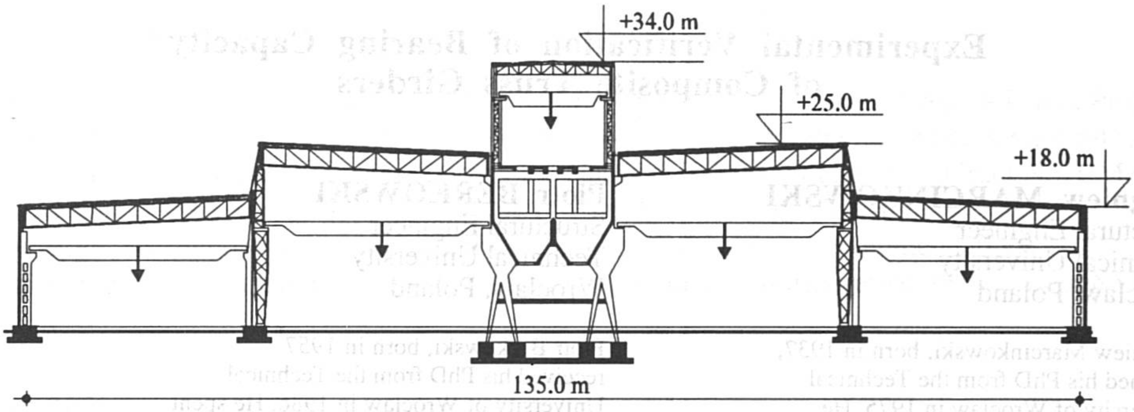

Fig. 1 Transversal section of the blocked five bay shop

Bearing capacity of the main structural elements of the roof decides the global safety of the object. Therefore it was recognized as advisable and necessary to get information about bearing capacity of these girders from another, independent source of information. To this aim the existing deflection of 72 girders was measured and that provided a sufficient number of tests for statistical elaboration of research results.

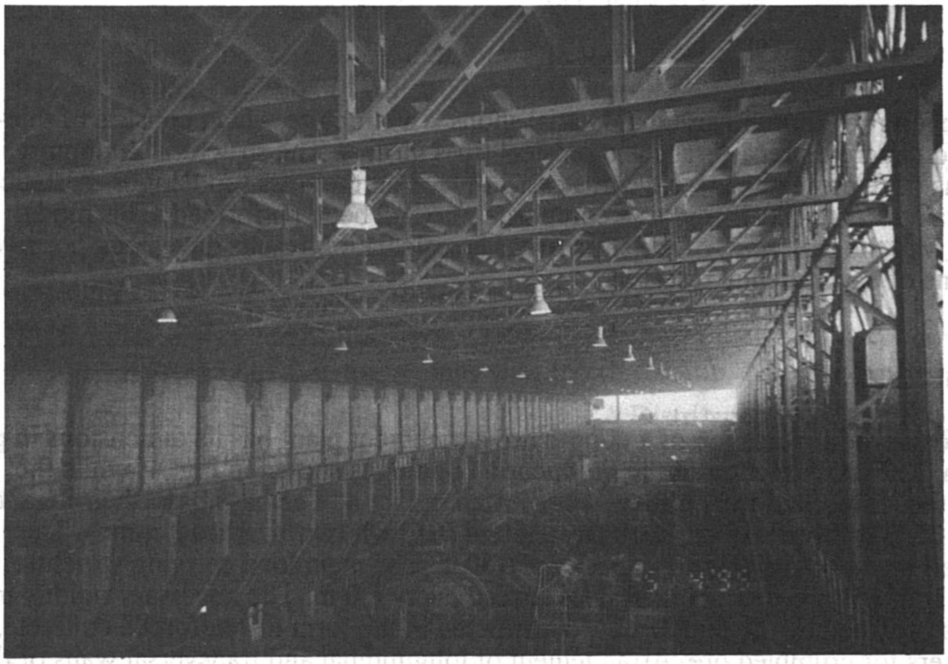

Fig. 2 General view of the researched roof truss girders

2. Principles of co-operation between steel and concrete in composite structures

Slender and thickset bent beams and columns are the most often type of composite steel-concrete structures used in building engineering. The static model of a compressed top flange of examined roof girder can be considered as a compressed composite section as shown in Fig. 3. In conformity with the Polish Code PN-91/B-03302 which takes into account the requirements of Eurocode 4 within the domain of composite action of concrete and steel, namely that the connection of steel with concrete should be assured if tangential stress τ appears in the contact surface between these two materials. This condition is satisfied beacuse the reinforced concrete curb (3) is mechanically linked to the batten plates (2).

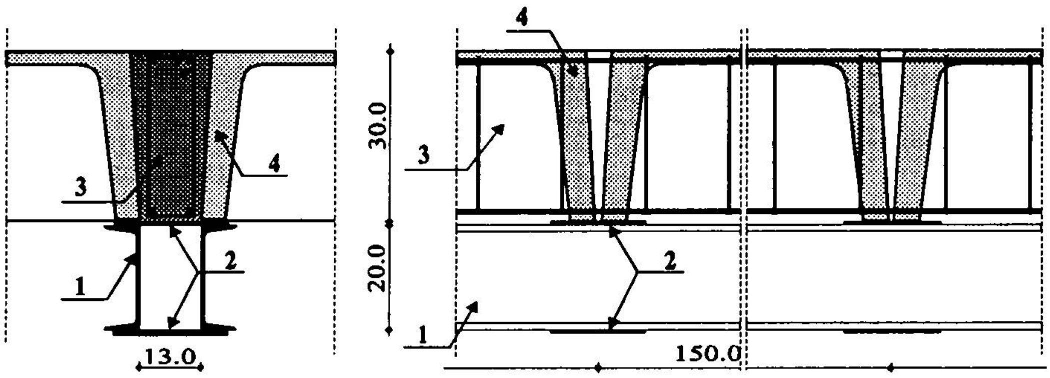

Fig. 3 Composite section of a truss girder top flange: (1) - steel top flange of a girder, (2) - batten plates connecting branches of a girder flange, (3) - R/C curb, (4) - precast thin-walled reinforced concrete slabs 30x150x587 cm

Connectors between steel and concrete are not necessary, if tangential stress τ in connection plane is less than 0.6 MPa. In our case tangential stress caused by:

- temperature difference,
- shearing forces and bending at intervals between nodes,
- concrete shrinkage

is $\tau < 0.8$ MPa. It signifies, that batten plates (2) - connectors between steel and concrete are not loaded. Bearing capacity N_{pl} of a composite section, according to PN-91/B-03302, is given by:

$$N_{pl} = F_s \cdot R_s + 0.85 \cdot F_b \cdot R_b + F_a \cdot R_a,$$

where: F_s, F_b, F_a - cross-section areas of steel, concrete and reinforcement; R_s, R_b, R_a - strength of steel, concrete and reinforcement. This equation was used while determining the state of stress and strain in the examined composite section, as part of a truss girder.

3. Calculation scheme of truss girder

A statically determined truss, as in Fig. 4., was accepted as the scheme for calculation.

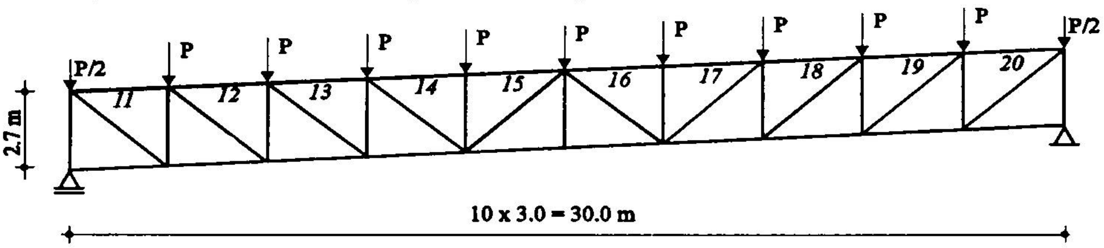

Fig. 4 Calculation scheme of truss girder

The force P is a dead load for which the girder's deflection line is calculated.
The girder works in two phases:

- in phase I the load has the value of $P_m = 32.09$ kN. This load works during assembling the roof elements and is carried only by steel girder;
- in phase II the load has the value of $P = 78.73$ kN. The composite steel-concrete section of truss girder top flange, as in Fig. 3, takes part in carrying the load from the level P_m to P.

Phases of truss girder work are illustrated in Fig. 5, that presents the relation between load and displacement: $P = P(f)$. A diagram presenting $P = P(\sigma)$ would be very similar. Introduction of an equivalent stiffness of truss girder would be useful for interpretation of information presented in Fig. 5. In phase I, that is to the load level corresponding to load P_m existing while assembling the structure, and the top girder works as a "pure" steel truss. The equivalent stiffness $(EJ)_1$ is proportional to tg α_1. During phase II, it is above the load level P_m existing while assembling, the girder works as a composite steel-concrete element. The equivalent stiffness

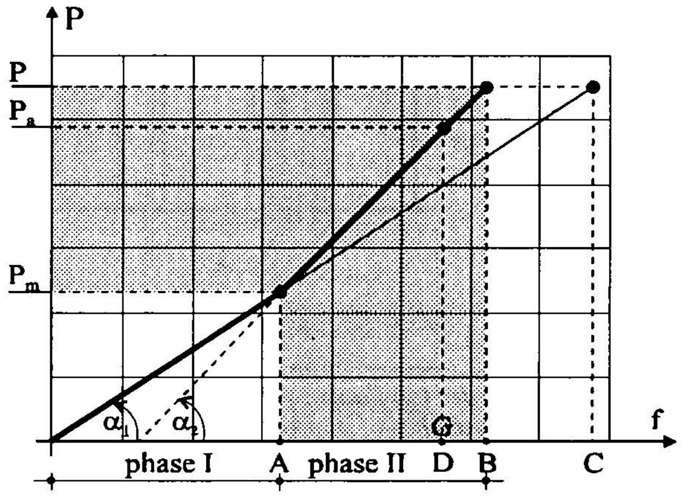

Fig. 5 Relation P = P(f) for phase I and II of a girder's work path. Notations: A, B, C, D - displacements of a middle point of a girder from: characteristic assembling load P_m, characteristic total load P - for a composite model, characteristic total load P - without accounting for concrete co-operation, characteristic real load P_a

$(EJ)_2$ of the composite section is proportional to tg α_2. It is worth stating that $(EJ)_2 > (EJ)_1$. In the examined problem $(EJ)_2 = 1.16 \cdot (EJ)_1$. It indicates that the girder equivalent stiffness after assembling increased about 16%.

To verify experimentally the presented calculation model, the deflection measurement of 72 truss girders were carried out. Deflection measurements were realized for real state of load P_a. The obtained mean deflection value of truss middle joint (point G in Fig. 5) precisely corresponds to the expected result delivered for a composite structure working in phase II.

4. Results of geodetic measurements of truss girders' deflections

The aim of geodetic measurements of vertical displacement of roof girders was an experimental verification of the supposition that examined roof girder works in two phases:

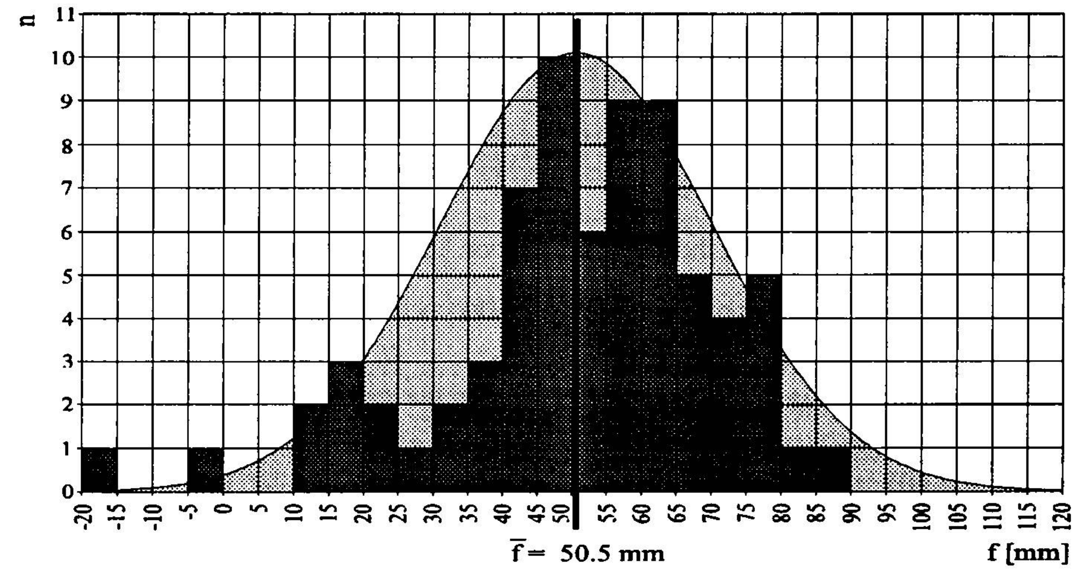

Fig. 6 Histogram and diagram of normal distribution of the geodetic measurement of roof girders' displacement

- phase I - the load level corresponding to load P_m existing while assembling - as a "pure" steel truss;
- phase II - above the load level P_m - as a steel-concrete composite structure.

Measurement of 72 truss girder deflection was carried out. The synthesis of measurement results is presented in Fig. 6 as a histogram.

A population of measurement results was divided into fractions of d = 5 mm width. Conclusive value of measured deflection is the arithmetic average of measurement results, because it is an unbiased estimator of expected value. Its value is $\overline{f} = 50.5$ mm, as shown in Fig. 6. The results of measurements, approximated by a normal Gauss distribution, have a following statistical parameter: $n = 72$, $\overline{f} = 50.5$ mm, $v = 39\%$, $\Delta = 19.75$ mm. Deviation of geodetic measurement from the average value of $\overline{f}$ defines a girder geometric production imperfection. This interesting problem was not developed in this work because of the lack of space.

5. Comparison of stress and deflection calculation results with experimental measurements

In Tab. 1 stresses expected from calculation to appear in the top flange of roof girder are presented for two models of girder construction:
- traditional model - structure works as a separate steel truss;
- real model - a top flange works as a composite steel-concrete section.

Tab. 1 Specification of force and stress in a composite steel-concrete top flange of a roof girder

Member number - Fig. 3	Member area	Assembling load P_m		Total load P		Equivalent area	Load in phase II (composite section)			
		force	stress	force	stress	of composite	force		stress	
	F_s	N_{sI}	in steel	N_s	in steel	section	N_{sII}	N_{bII}	in steel	in concrete
	cm²	kN	MPa	kN	MPa	cm²	kN	kN	MPa	MPa
11	57.4	-187	**-32.6**	-482	**-84.0**	108.11	-157	-138	**-59.9**	**-3.1**
12	57.4	-331	**-57.7**	-853	**-148.7**	108.11	-277	-245	**-106.0**	**-5.4**
13	74.0	-432	**-58.4**	-1113	**-150.4**	124.71	-404	-277	**-113.0**	**-6.2**
14	98.0	-490	**-50.0**	-1262	**-128.7**	148.71	-509	-263	**-101.9**	**-5.8**
15	98.0	-490	**-50.0**	-1262	**-128.7**	148.71	-509	-263	**-101.9**	**-5.8**
16	98.0	-476	**-48.5**	-1225	**-125.0**	148.71	-494	-255	**-98.9**	**-5.7**
17	98.0	-476	**-48.5**	-1225	**-125.0**	148.71	-494	-255	**-98.9**	**-5.7**
18	74.0	-404	**-54.5**	-1039	**-140.4**	124.71	-377	-258	**-105.5**	**-5.7**
19	57.4	-288	**-50.2**	-742	**-129.3**	108.11	-241	-213	**-92.2**	**-4.7**
20	57.4	-130	**-22.6**	-334	**-58.2**	108.11	-108	-96	**-41.5**	**-2.1**
Area of concrete section - $F_b = 450$ cm², $E_b = 25.3 \cdot 10^3$ MPa, $E_a = 2.05 \cdot 10^5$ MPa										

Taking into consideration the co-operation between concrete and steel in phase II, that is above the level of assembling load P_m - the stress in steel section, e. c. for the member 13, are reduced from 150.4 to 113.0 MPa, that is nearly 25%. These reduced stresses are "consumed" by the concrete part of the top flange, where the stress in concrete attains a maximum value of 6.2 MPa.

These reduced stresses in steel section in the composite structural model are accompanied by the adequate lower displacement. Deflection charts for considered load levels are shown in Fig. 7.

Taking into consideration the composite structural model, a perfect consistency between the calculation results and geodetic measurements was obtained.

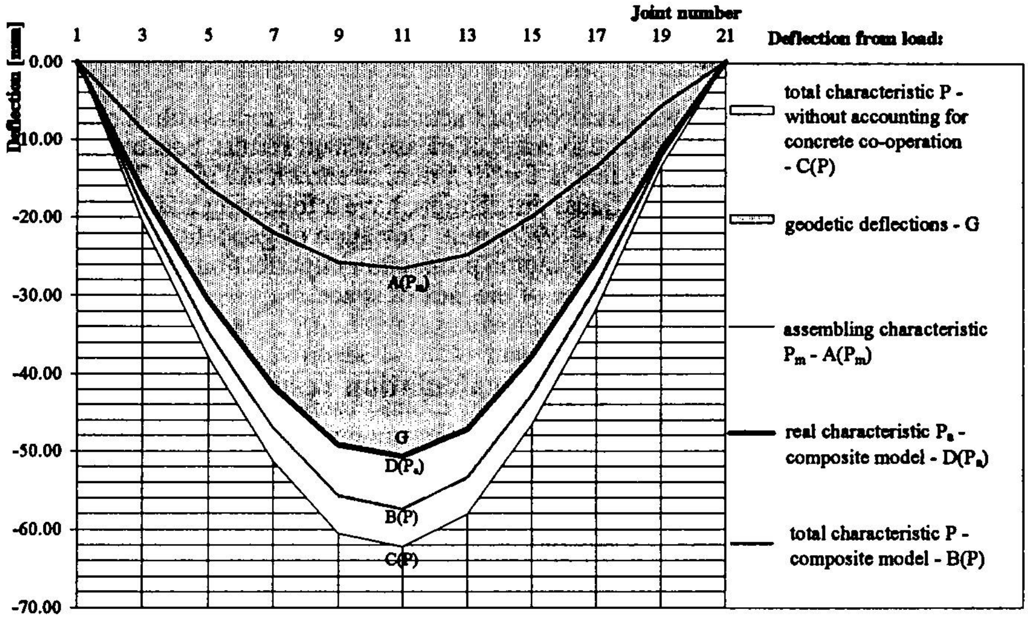

Fig. 7 Calculated and measured displacements of roof girders for considered load levels

In Fig. 7 the points G and D(P_a) coincide, and in Fig. 5, under the real load level P_a, the mean value of the displacement - point G - is obtained.

6. Recapitulation and conclusions

Results of computational static and strength analysis made for the steel roof girders of 30 m span and results of geodetic measurements of vertical displacements of girders' middle joints have been presented in this paper. The latter was treated as an independent second source of information in order to verify the postulate concerning existence of a co-operation between the steel girders' top flange and the concrete curb connected to it.

A classical structural solution of precast thin-walled reinforced concrete slabs supported on upper flange of steel truss girders is the placing of a concrete curb in phase I of construction work. In this phase, i. e. till the load reaches the value of the so-called "assembling" load P_m, the girder works as a "pure" steel truss. In phase II, the concrete curb after hardening and mechanical connection to the girders' top flange, starts work in carrying the load. Girder top flange then works as a composite section, delivering a 16% capacity reserve. The "two-phase" calculation model was positively verified by the geodetic measurements of girder displacement.

The presented example of experimental verification of steel roof girders justifies the formulation of a postulate to introduce the notion of "experimental bearing capacity" into the building codes, so that the research results on deflections and stresses would be equally legalized with the results of static and strength calculations, especially in the case of making a diagnosis of existing structure technical status.