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Composite deck construction for the rehabilitation of motorway bridges

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Abstract. Traffic decks of steel or composite motorway bridges sometimes provide the opportunity of using the composite action between an existing steel deck and a reinforced concrete plate (RC plate) in the process of rehabilitation, i.e., to increase the load-carrying capacity of the deck for concentrated traffic loads. The steel decks may be orthotropic decks or also unstiffened steel plates, which during the rehabilitation are connected with the RC plate by shear studs, such developing an improved local load distribution by the joint behaviour of the two plate elements. Investigations carried out, both experimentally and numerically, were performed in order to quantitatively assess the combined static behaviour and to qualitatively verify the usability of the structure for dynamic loading. The paper reports on the testing, the numerical simulation as well as the comparison of the results. Conclusions drawn for practical design indicated that the static behaviour of these structures may be very efficient and can also be analysed numerically. Further, the results gave evidence of a highly robust behaviour under fatigue equivalent cyclic traffic loading.

Key words: composite structures; traffic decks; orthotropic steel plates; RC plates; rehabilitation; motorway bridges; steel bridges; composite bridges.

1. Introduction

In the course of the rehabilitation of a motorway bridge near Salzburg in Austria investigations towards an improved load-carrying behaviour of the bridge deck led to a specific composite construction in form of the orthotropic steel deck combined with a RC slab.

This motorway bridge of the north-south transit route is one of the most heavily loaded road bridges of this area. Up to an average of 7000 trucks daily and 200 special vehicles yearly with up to 240 tons each, the load impact on the traffic deck has been continuously increasing since 1970, when this steel bridge in form of a “middle arch structure” was built as the first example of this type of structural system (Beer and Müller 1970). It is a structure with a bow string arch in the center line with a span of 133 m and the traffic deck projecting to both sides over 15 m each (Fig. 1).

The deck construction originally was an orthotropic steel deck with an asphalt layer on it, which was covered by a concrete surface (Fig. 2). As result of the highly increased impact of the traffic load compared to the construction time the deck surface has been damaged in the form of cracks in the concrete slab, which allowed penetration of the salt-contaminated surface water seriously impairing the

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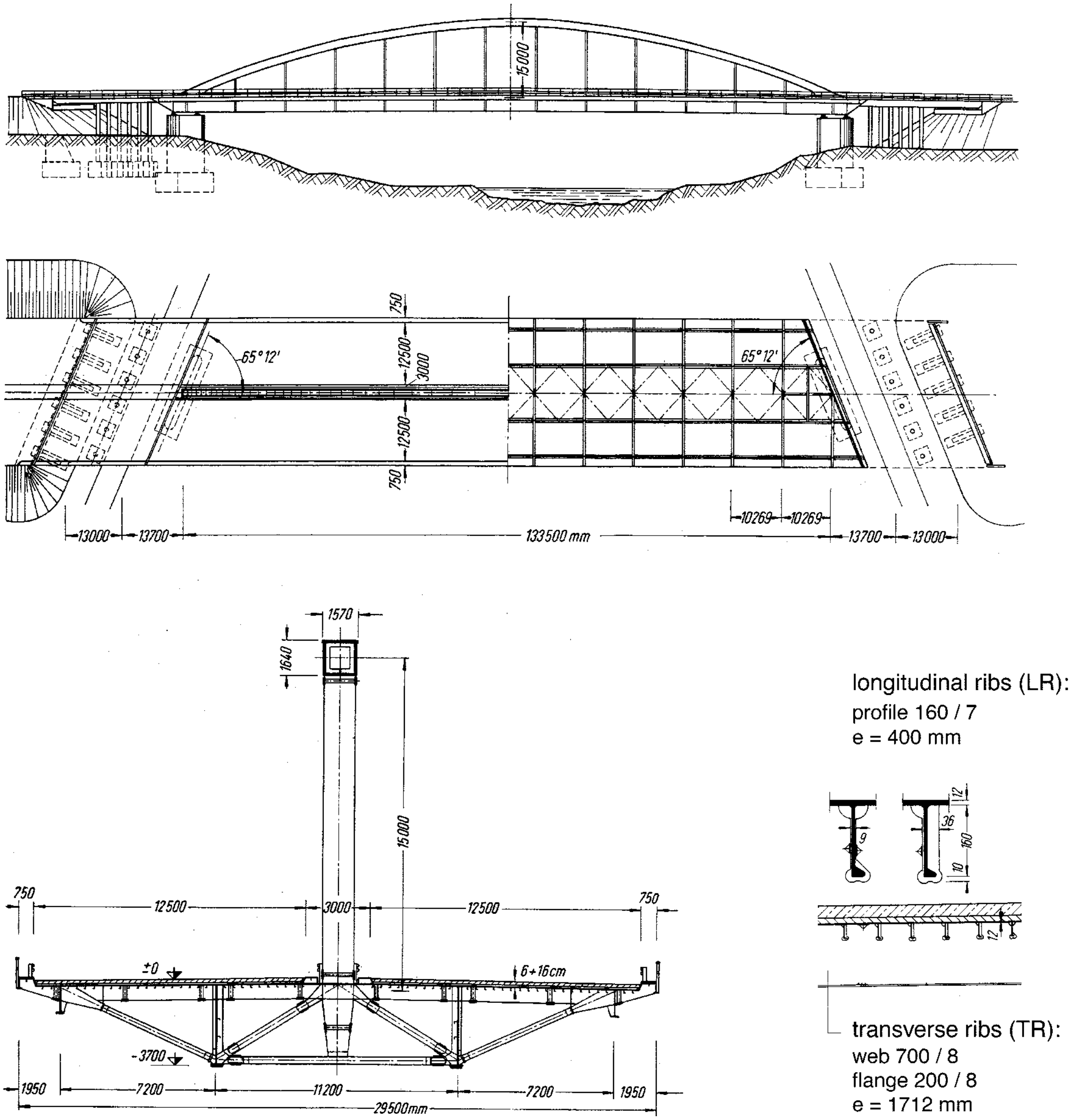

Fig. 1 Structural system of the motorway bridge

consistency of the asphalt layer.

The rehabilitation of the traffic deck has then been based on the idea of re-arranging the two layers on top of the steel deck, the first now being the RC slab and the second the asphalt surface. The RC slab could now be connected with the steel deck by stud bolts resulting in a composite action (Fig. 3). In this composite construction two plates of very different stiffness have to work together; - the RC plate should locally distribute the high concentrated traffic loads over the orthotropic steel plate, which then should transfer them to the main girders of the bridge.

Similar considerations have also been made for the rehabilitation of other motorway bridges showing

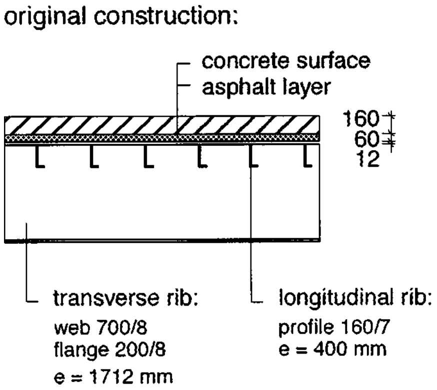

Fig. 2 Original deck construction

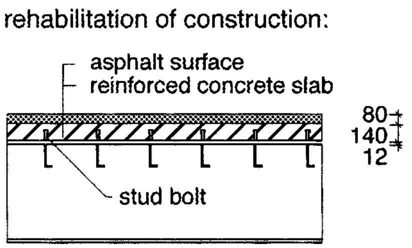

Fig. 3 Rehabilitation of deck construction

difficulties with the load-carrying behaviour of the traffic deck, e.g., the deck of the Rio-Niteroi Bridge in Brazil (Battista and Pfeil 1999).

In the following a report is given on the experimental and numerical investigations carried out for the composite action of decks consisting of RC slab and steel decks of different construction type. The basic part of the study was focused on the behaviour of the orthotropic steel deck of the given bridge with the objective of developing a physically sound design basis for the rehabilitation. For this purpose both numerical as well as experimental work was carried out in order to use the latter for the validation of the numerical simulation and to prove the robustness of the deck under dynamic action. The variation of the composite action - with or without shear - studs served for obtaining an overview on the different load-carrying capacities.

A similar study was added for an unstiffened steel deck in order to widen the knowledge for other applications by use of the same test model and test equipment.

2. Composite construction with orthotropic steel deck

2.1. Structural modelling of the traffic deck

For structural design purposes the traffic deck was transferred into an analysis model, which describes the deck structure by a submodel embedded in the global model of the main bridge structure (Fig. 4). While the global model represents the composite deck construction by an orthotropic plate, the submodel discretely models the deck construction by an elastic RC plate and a steel deck plate, to which the longitudinal and transverse ribs are eccentrically connected in form of beam elements. RC plate and steel deck plate are rigidly connected. The numerical analysis was based on the ABAQUS program (Hibbitt, Karlsson & Sorensen, Version 5.8).

For reasons of a realistic assessment of the load-carrying behaviour of the composite traffic deck a testing program was performed, which then was connected with comparative numerical simulations in order to develop a numerical design model. The test model should have the scale 1:1 for reasons of reproducing a realistic concrete behaviour. Due to the restrictions of the laboratory the size of the model had to be reduced to a local part of the deck (see Fig. 4). Since in this way the detailed investigations are directed to a cut-out of the deck structure, the boundary conditions of the local model had to be adapted to the continuity of the larger submodel. This was achieved in good approximation through supporting

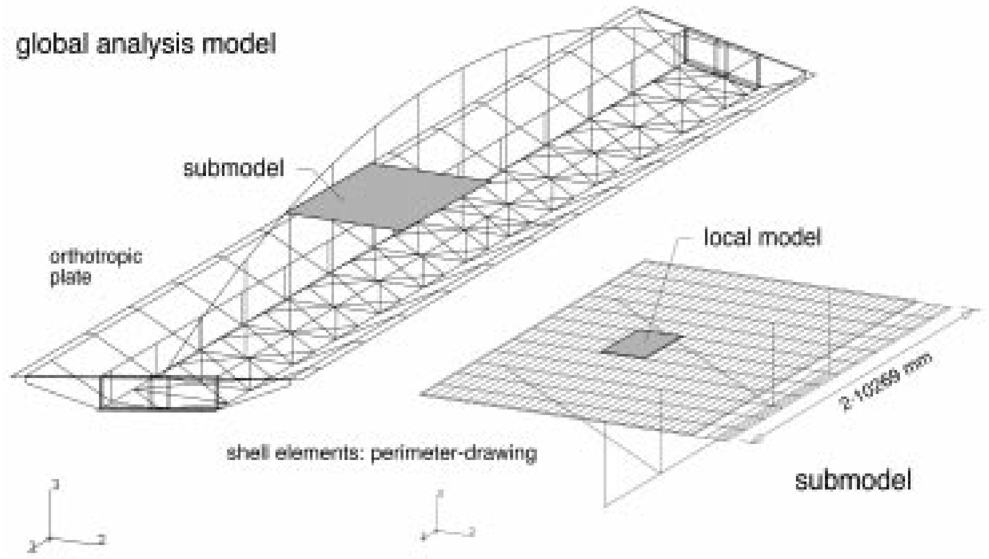

Fig. 4 Analysis model of the traffic deck

the boundary by specific girders, whose rigidity represents the overall flexibility of the surrounding deck structure.

2.2. Experimental investigations

2.2.1. Test model

The local test model consisted of a rectangular part of the deck (2.0×3.4 m) and included 4 longitudinal and 3 transverse ribs; the longitudinal boundaries rested on supporting girders described above (Fig. 5). The dimensions in detail are given in Tables 1 and 2.

The steel part was made of steel S235 with an actual yield strength of 280 N/mm^2. The strength class of the concrete was C55/65 in one case (Test Model 1) and C40/50 in the other (Test Model 2). The mean compressive cube strength (20 cm) of the concrete was 61 N/mm^2 for Test Model 1 and 40 N/mm^2 for Test Model 2, both measured after 40 days. The reinforcing steel was of the yield strength 550 N/mm^2. The shear studs were attached in a pattern following the webs of the longitudinal ribs and the supporting girders.

The steel structure was supported at the 4 corners by elastomere-pads. The steel deck of the test model including the reinforcement is illustrated by Fig. 5.

2.2.2. Traffic load

The test loading was defined by local wheel-loads derived from the load models given by EUROCODE 1-3 (Eurocode 1, 1995). The dominating local load followed from Load Model 2 represented by an axle load of 400 kN, i.e., two wheel loads of 200 kN with a spacing of 2 m (Fig. 6). The loaded base area was approximated by 400×400 mm, the load was introduced into the RC plate via a steel plate.

The test loading was carried out statically as well as dynamically. The dynamic loading was defined by load cycles of ΔP = 90 and 140 kN as described later in detail.

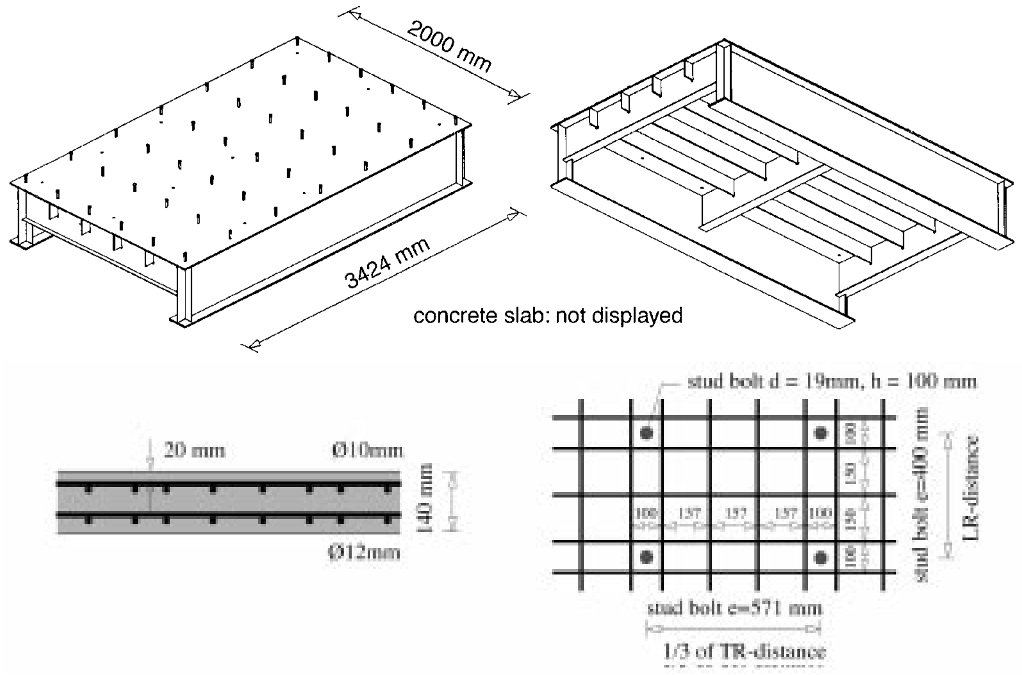

Fig. 5 Steel deck and reinforced concrete slab of test model

Table 1 Dimensions of test model [mm]

Steel deck plate: thickness	12 mm
Transverse rib: web	342/8 mm
Transverse rib: flange	100/8 mm
Longitudinal rib	160/12 mm
Distance of transverse ribs	1712 mm
Distance of longitudinal ribs	400 mm

Table 2 Reinforced concrete slab

Concrete slab:		Thickness 140 mm	
Reinforcement:	diameter	pieces per length	cm^2/m
Top, longitudinal	10 mm	3/400 mm	5.89
Top, transverse	10 mm	4/571 mm	5.51
Bottom, longitudinal	12 mm	3/400 mm	8.48
Bottom, transverse	12 mm	4/571 mm	7.93

2.2.3. Testing procedure

The testing procedure was performed for quasi-static loading and for cyclic loading with frequencies between 3 and 5 Hz. The loading started for Test Model 1 with the static load up to 200 kN applied in sequential steps of 50 kN. Then the cyclic loading followed with 2 million cycles of ΔP = 90 kN and an upper load of 100 kN, applied in sequential steps of 125000 cycles and each step was followed by a

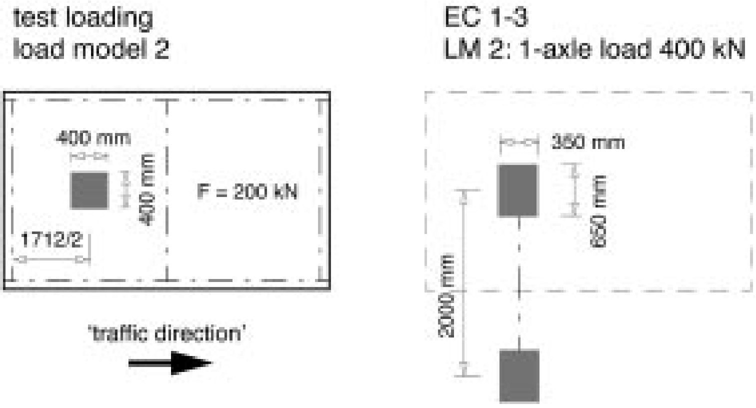

Fig. 6 Test loading

reference measurement for the static load 200 kN. The next part of the cyclic loading with 1 million cycles was performed with ΔP = 140 kN and an upper load of 200 kN in sequential steps of 250000 cycles, again each step was followed by a reference measurement for the static load 200 kN. Therefore, in total 3 million cycles were applied.

The Test Model 2 was performed with the same steel deck after the concrete plate had been removed and the welds of the shear studs had been tested. The test procedure started with the quasi-static loading in the same way as with Test Model 1. Again 3 million load cycles were applied with reference measurements after each step, however, the sequential steps were larger, i.e., about 400000 cycles. Then 1.8 million cycles were performed with ΔP = 90 kN and 1.2 million cycles for ΔP = 140 kN.

After the dynamic loading the static load was stepwise increased up to 980 kN. At the load level of 740 kN a discontinuity of the measurements appeared, which indicated that the bonding between concrete surface and steel plate had been brought to an end. This will be discussed later with the results of the testing program.

After reaching the maximum of 980 kN additional static and dynamic load sequences (450000 cycles) were applied in order to investigate the different behaviour. Unfortunately only limited measurements could be taken due to the overstressing of the measurement equipment through the maximum load.

Test Model 3 used the RC plate and steel deck of previous Test Model 2 but excluding any effect of bonding or real composite action. For this purpose the shear studs had been made inactive by drilling them loose from the concrete slab. The only composite action between steel deck and concrete plate

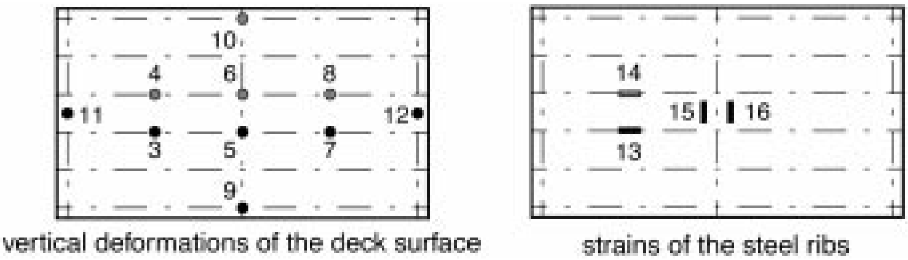

Fig. 7 Locations of test measurements for stiffened deck

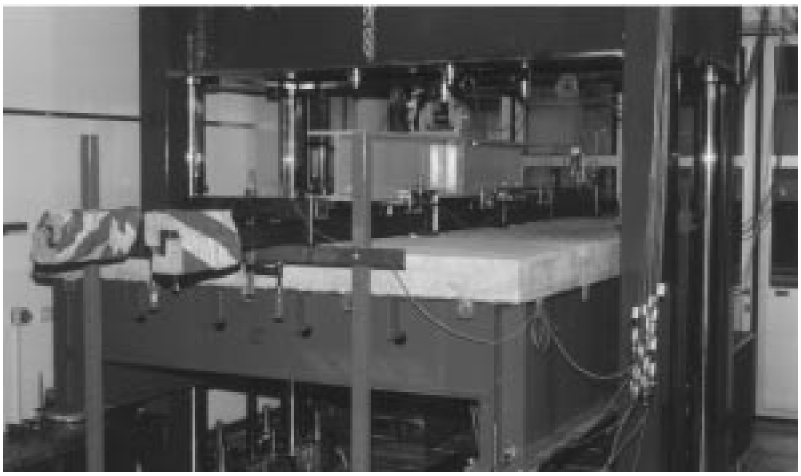

Fig. 8 Test model in laboratory

occurred from friction. The test procedure in this case was restricted to quasi-static loading, which was applied stepwise up to 980 kN in analogous way as for Test Model 2 before.

Test Model 4 consisted of the steel deck alone and was just used for comparative reasons. After removing the concrete plate quasi-static loads were applied in steps up to 600 kN.

The measurements consisted of vertical deformations of the deck surface, of differential deformations between the horizontal planes of the steel plate and the RC slab and further of strains at the lower edges of the steel ribs (Fig. 7). The test arrangement in the laboratory is shown in Fig. 8 (Kernbichler 1998).

2.3. Numerical simulation

2.3.1. Numerical model

The idealized structural model representing the local test model is illustrated in Fig. 9. The structural steel components were modelled by means of shell- and beam-finite-elements. The RC slab and the deck plate were represented by shell elements. The webs of the longitudinal and transverse ribs as well

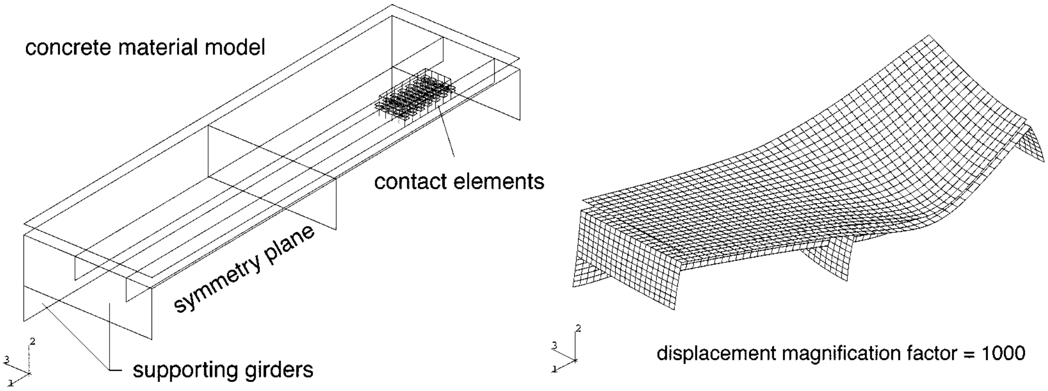

Fig. 9 Local analysis model

Table 3 Values of material [N/mm^2] used for the numerical Model 2

E-modulus of steel	190000
Yield strength of steel f_y	280
E-modulus of concrete	27000

as the supporting edge-girders were also modelled by shell elements and the flanges by beam elements.

The condition of symmetry has been used for the numerical model as illustrated in Fig. 9. The vertical supports were applied at the four end points of the supporting girders. The horizontal restraint was applied in the center point of the steel deck plate.

The steel deck was described by an elastic-plastic material model with the parameters given in Table 3.

The RC plate was represented by a material model provided by ABAQUS, which accounts for the specific behaviour of different strengths in compression and tension, of cracks in the tension zone and of the tension stiffening effect of reinforced concrete. Reinforcement as well as cracks are considered in “smeared out”-form and are defined in specific layers depending on the direction of the plate. The modulus of elasticity is given in Table 3 corresponding to the material C 40/50 of Test Model 2.

The parabolic stress-strain curve for uniaxial compression of concrete was approximated in polygonal form (Fig. 10). The biaxial strength parameters are given in Table 4 based on concrete C 40/50 with the characteristic compressive strength f_{ck} = 40 N/mm² and the mean tensile strength f_{ctm} = 3.5 N/mm^2. The rest of the parameters followed from the model of ABAQUS.

The limiting strength curve of the concrete is illustrated in Fig. 11.

The composite action between RC slab and steel deck was investigated by various assumptions, i.e., discrete shear studs of different stiffness, continuous rigid composite action or pure contact without any composite action. In the latter case the transverse connection between concrete slab and steel deck was modelled by contact elements, in particular in the zone of the vertical load introduction.

2.3.2. Calculation of test-load cases

The numerical simulations were made for all the 4 Test Models at the discrete steps of the static loading. They produced the deformations and stresses for the purpose of comparison with the test results at the corresponding locations (Greiner 1998).

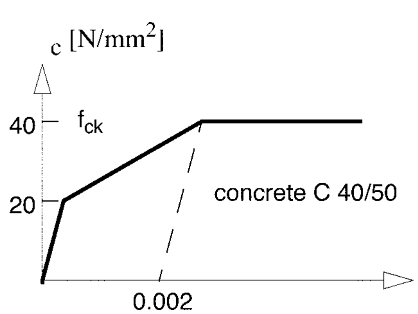

Fig. 10 Stress-strain curve of concrete

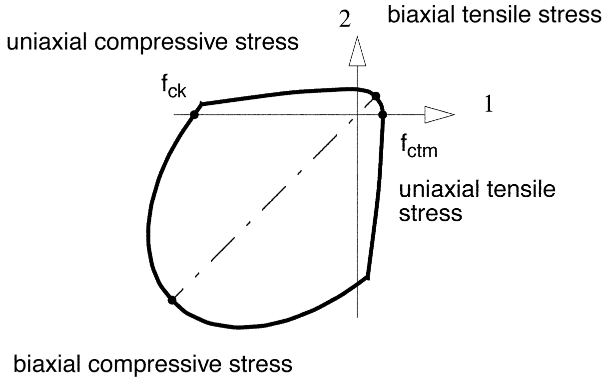

Fig. 11 Limiting strength curve of concrete

Table 4 Parameters for definition of limiting strength curve

2-axial / 1-axial compressive strength	1.16
1-axial tensile strength / 1-axial compressive strength	0.0875
2-axial / 1-axial plastic strain	1.28
Tensile strength at compression / 1-axial tensile strength	0.33

2.4. Results and comparison

2.4.1. Model with rigid composite action

The experimental and numerical data are represented in Table 5 for the most significant locations at the load level of 200 kN. The measured deformations refer to the beginning and to the end of the cyclic loading; the measured strains did not show quantitatively perceptible alterations.

An overview on the calculated deformed structure is given in Fig. 12 under a wheel-load of 200 kN.

The results illustrate that the effect of the two concrete classes is practically insignificant. The relatively higher deformations of Test Model 1 versus Test Model 2 can be explained by pre-loadings with another load model in the first case. Among the numerically investigated alternatives of the effect of the composite action the closest accordance with the test results was obtained for the assumption of

Table 5 Test Model 1 and 2 with rigid composite action for wheel-load of 200 kN

Location of measurement	Vertical deformation [mm]			Stress [N/mm^2]		
	3, 4	5, 6	7, 8	13, 14	15	16
Test model (1)†	0.76	0.54	0.30	40	32	26
Numerical result	0.67	0.48	0.24	36	23	26
Test model (1)*	0.80	0.56	0.31			
Test model (2)†	0.70	0.50	0.26	37	27	30
Numerical result	0.68	0.49	0.24	38	23	27
Test model (2)*	0.78	0.56	0.30			

† … beginning of cyclic loading
* … end of cyclic loading

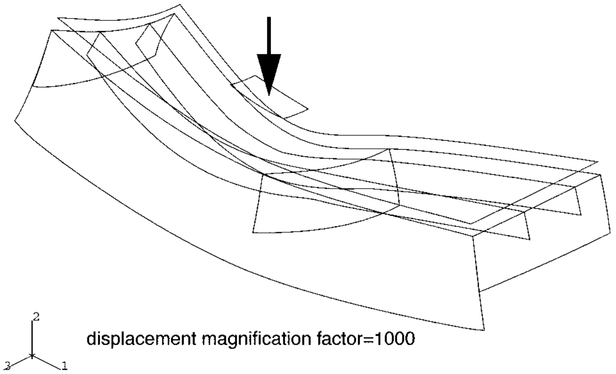

Fig. 12 Model 2 with rigid composite action

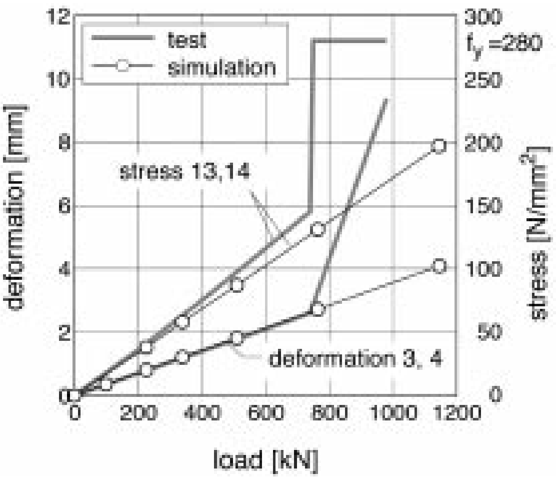

Fig. 13 Load-deformation curve and load-stress curve

rigid composite action; consequently these results were put into Table 5.

The tests further verified that the fatigue effect in the composite action due to about 3 million load cycles with rather high ΔP of 90 and 140 kN was not significant, resulting in an increase of deformations up to about 10% only.

Further to the tests with the EUROCODE-load model 2 increased load steps were investigated up to 980 kN (see Fig. 13). The behaviour to be observed was linear up to the load step of 740 kN, when the test indicated accoustically as well as by a jump of the differential horizontal deformations between concrete plate and steel deck, that the bond strength in the contact plane had been exceeded. The curves in Fig. 13 illustrate this effect by an increased inclination at higher load steps. Additional tests with Test Model 2 at the load level of 200 kN after the above overloading up to 980 kN showed differential deformations of about 0.10 to 0.15 mm compared to almost zero at the load steps lower than 740 kN. Regarding the laterally free edges of the RC plate and the local behaviour of the Test Model in relation to the full traffic deck it may be concluded that such differential deformations between RC plate and steel deck will actually be significantly restrained by the surrounding concrete slab. Therefore, it may be expected that under local wheel-loads good composite action is also provided by the shear studs alone without taking into account the additional bonding.

Thus, the given behaviour may be classified as sufficiently robust for use under traffic loading according to EUROCODE. It not only showed that 3 million load cycles could not significantly impair the load-carrying behaviour, but also that a very stiff composite action was maintained up to a load intensity higher than 3.5 times the standard wheel-load.

2.4.2. Model without composite action

Since Test Model 2 indicated the different load-carrying behaviour due to locally reduced adhesive strength the investigation has been extended to Test Model 3 without any regular composite action, i.e. without the action of shear studs and without bonding. The concrete slab was loosely put on the steel deck, so that just friction could affect the “in-plane interaction” of concrete plate and steel deck. The test loading was identical to the statical tests before and load steps were increased up to the limit load, which appeared as punching failure of the concrete slab in the range of 600 to 800 kN. (On the contrary,

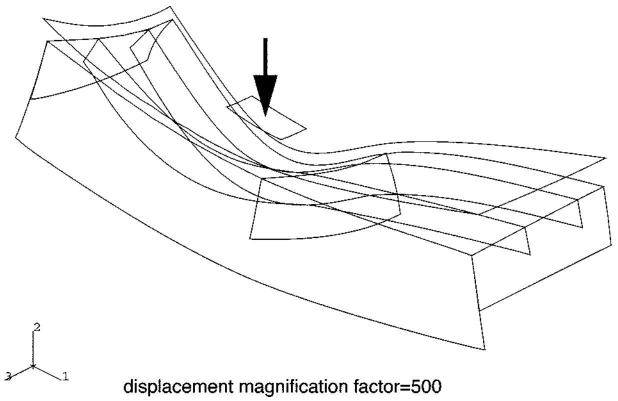

Fig. 14 Model 3 without composite action

Table 6 Test Model 3 without composite action for wheel-load of 200 kN

Location of measurement	Vertical deformation [mm]			Stress [N/mm^2]		
	3, 4	5, 6	7, 8	13, 14	15	16
Test	1.40	0.97	0.50	67	29	36
Numerical result	1.49	0.87	0.28	75	39	49

Test Model 2 didnt show a punching failure, even not at the limit of 980 kN.)

The experimental and numerical stresses are compared in Table 6 at the load-level of 200 kN. The accordance between test and calculation is good, in particular if the imponderable friction-effect is considered.

The calculated deformations at the same load-level are illustrated by Fig. 14 showing that the concrete plate lifted up in certain areas of the deck. The comparison of the measured and calculated deformations (Table 6) contains approximations due to the previous overloading of the elastomere pads of the supports, which affected the accuracy of the deformation measurements.

By the comparison of Test Model 2 and 3 the beneficial effect of the composite action on the local load-carrying behaviour could be well evaluated. The maximum deformations and stresses of the longitudinal ribs differ by a factor of approximately 2 for a wheel-load of 200 kN. Understandably, the least stressed elements, e.g., the transverse rib, are not so highly affected.

Further, it could be derived from the higher load steps above 200 kN that the composite action of Test Model 2 produced a linear, load-proportional increase of stresses and deformations, while the friction-related action of Test Model 3 resulted in a progressive increase.

2.4.3. Model without RC plate

The fourth test (Test Model 4) was carried out for the steel deck alone, without concrete slab. It served for completion of the study on the local load-carrying behaviour by considering this utmost limiting case.

The results for a wheel-load of 200 kN are presented in Table 7; for the measured deformations the same applies as stated above for Test Model 3.

The test results for the stresses were linearly recalculated from the measured strains even if they were

Table 7 Test Model 4 without RC slab for wheel-load of 200 kN

Location of measurement	Vertical deformation [mm]			Stress [N/mm^2]		
	3, 4	5, 6	7, 8	13, 14	15	16
Test	3.75	1.65	0.13	295	76	98
Numerical result	4.51	1.45	0.12	280	77	115

in the inelastic range.

The comparison illustrates that the deck without concrete slab would significantly be overstressed for a wheel-load of 200 kN.

2.4.4. Summary

The test program of the local cut-out of the reinforced concrete-orthotropic steel traffic deck dealt with the different load-carrying models of rigid composite action including full bonding between concrete plate and steel deck, of composite action without bonding, of reduced composite action just by friction and of the steel deck alone. The investigation was focused on the capacity and behaviour of the local load-distribution of the traffic deck as well as on the usefulness of numerical modelling with respect to their applicability to practical bridge structures.

The comparison of all the four models is illustrated by the local stresses of the longitudinal ribs in Fig. 15. It firstly shows the highly beneficial effect of the load-distribution due to the RC plate, in particular for rigid composite action. The rigid composite action with bonding was active up to more than 3.5 times the maximum of the required wheel-load by EUROCODE. Even without regular composite action a good distributing behaviour may be achieved. Secondly, it could be proved by the testing that the impact of dynamic loading up to 3 million load cycles did not significantly affect the load-carrying behaviour, such confirming the robustness of the structural system. And thirdly, the investigation made clear that fairly good accordance between actual behaviour and numerical simulation may be achieved by existing computer codes and appropriate assumptions of the material data.

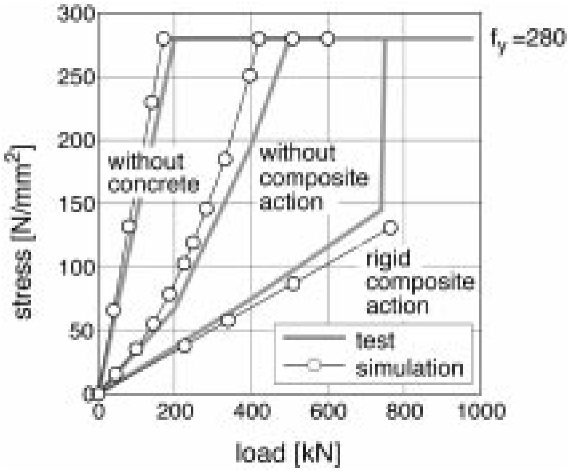

Fig. 15 Comparison of load-stress curves

3. Composite construction with unstiffened steel deck

3.1 Experimental investigations

Following the tests of the composite orthotropic deck described in section 2 the steel deck was further used for investigations of a composite construction with an unstiffened steel deck. For this purpose the longitudinal ribs were cut off the existing deck, so that just the steel deck plate remained between the transverse ribs. The shear studs were tested with respect of the welds and then the reinforced concrete plate was fabricated in the same form as for Test Model 2 before. The strength class of the concrete was C40/50 and the *E*-modulus resulted in 24200 N/mm^2. The mean compressive strength of the concrete was 39 N/mm^2.

This model was called Test Model 5 and apart from the unstiffened deck plate it was identical with Test Model 2, in particular with respect to the support conditions, the loading arrangement and the measurements. The locations of the measured deformations and strains are illustrated in Fig. 16. The vertical deformations were measured at the upper surface of the concrete plate, the strains at the lower surface of the steel deck plate and at the sides of the flanges of the transverse rib.

The loading was applied in quasi-static load steps up to 700 kN, when the bonding in the horizontal plane between concrete plate and steel deck came to an end again. The loading was then repeated for the new condition without bonding and resulted in increased deformations. During this second load sequence the concrete plate failed by punching below the concentrated load.

The model which had lost bonding in the half where the load was applied and had also undergone a punching failure there was further used for testing in the opposite half which had remained intact during the procedure. This means that a fairly original condition existed there, while the neighbouring part had lost bonding. This was called Test Model 6.

The stepwise loading of Test Model 6 then produced deformations under the wheel-load modestly

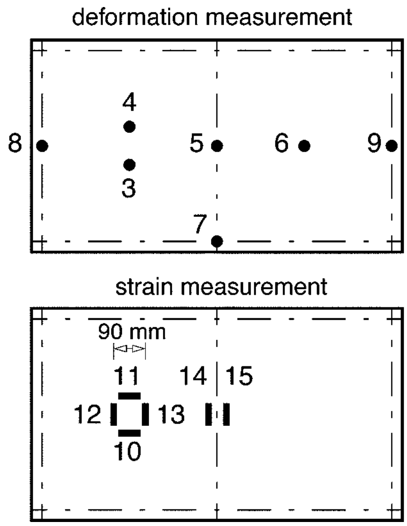

Fig. 16 Location of measurements for unstiffened deck

increased in comparison with those of Test Model 5 in the original condition of full bonding. During the load sequence the bonding came to an end at about 580 kN also in this area and the concrete plate again failed by punching failure.

Further testing of the Test Model without the composite action of the shear studs could not been carried out since after drilling free the studs the loose concrete plate due to the two punching failures did not allow a clear load application any more.

3.2. Numerical simulations

Numerical calculations were carried out for the unstiffened model in analogy with the calculations performed for the stiffened deck in section 2 before. Calculations were made for the assumption of rigid composite action for comparison with Test Model 5 and 6 and further for the assumption without any composite action. As mentioned above, for the latter no comparative test existed.

3.3. Results and comparison

3.3.1. Model with rigid composite action

The test results of the numerical calculation with rigid composite action are illustrated by the deformed structure in Fig. 17 as well as by the data in Tables 8 and 9 for a wheel-load of 200 kN.

The comparison with the test results of Test Model 5 (with full bonding) is given by Figs. 18 and 19 for the whole load sequence. The comparison shows the good accordance between test and analysis up to the load level of 700 KN, where the bonding strength was exceeded. The deformations below the concentrated load were just insignificantly higher than those for the stiffened deck construction, while the stresses there had quite different magnitude due to their

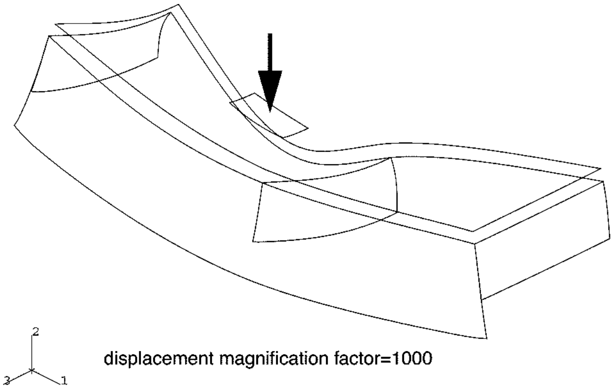

Fig. 17 Model 5 with rigid composite action

Table 8 Test Model 5 with rigid composite action for wheel-load 200 kN

Location of measurement	Vertical deformation [mm]					
	3	4	5	6	7	8
Test	0.77	0.77	0.52	0.28	0.31	0.36
Numerical result	0.82	0.82	0.55	0.24	0.31	0.32

Table 9 Test Model 5 with rigid composite action for wheel-load 200 kN

Location of measurement	Stress [N/mm^2]					
	10	11	12	13	14	15
Test	18.0	17.4	17.2	19.4	21.3	25.8
Numerical result	18.3	18.3	20.2	19.9	26.1	29.2

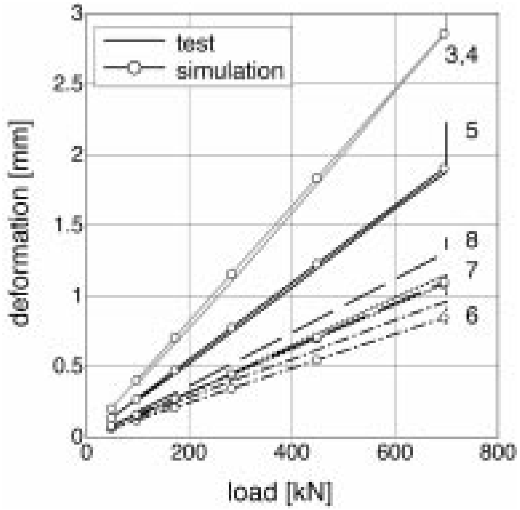

Fig. 18 Load-deformation curve

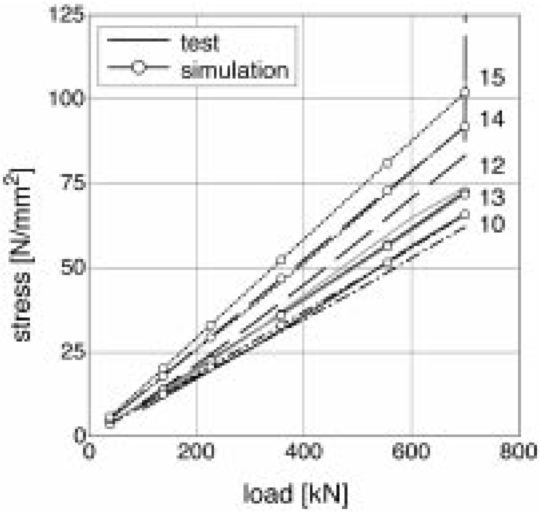

Fig. 19 Load-stress curve

different nature as stresses in the plate instead of the longitudinal ribs. However, the stresses in the transverse rib were just the same as in the stiffened condition, which together with the nearly equivalent deformations illustrates the good local load-distributing effect of the RC plate

Table 10 Test Model 5 without bonding for wheel-load of 200 kN

Location of measurement	Vertical deformation [mm]					
	3	4	5	6	7	8
Test	1.70	1.70	0.67	0.29	0.37	0.57

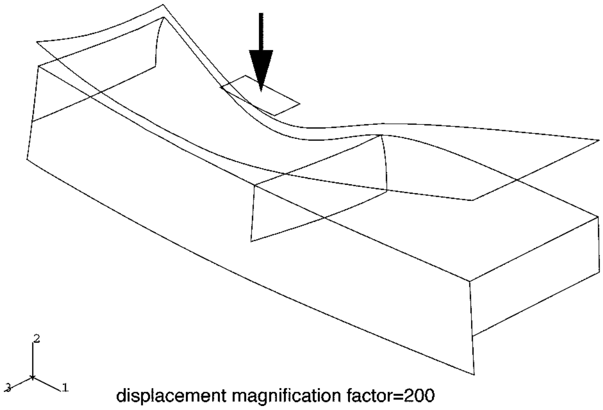

Fig. 20 Model 7 without composite action

connected with the steel plate.

The test results for Test Model 5 without bonding are given by the vertical deformations presented in Table 10, illustrating the ductile behaviour of the shear studs and the resulting effect on the load-carrying behaviour of the structure after bonding had been exceeded.

The behaviour of Test Model 6 with the bonding exceeded in the opposite half of the deck is just expressed by the vertical deformations below the wheel-load, which were 1.0 mm for 200 kN. This result compared with Fig. 18 indicated that the structure behaved a little bit more flexible, which is well in line with the physical understanding.

3.3.2. Model without composite action

The case of the unstiffened steel deck combined with a RC plate without any regular composite action could not be investigated by testing, however, it was analysed by a numerical simulation (Model 7). The deformed structure is demonstrated in Fig. 20 and the numerical data are presented in Table 11 and 12, both for the load-level of 200 kN.

Compared to the results of Test Model 5 and 6 the uplifting of the concrete plate is obvious as well as the expected higher vertical deformations below the wheel-load. On the other hand the stresses in the

Table 11 Model 7 without composite action for a wheel-load of 200 kN

Location of measurement	Vertical deformation [mm]					
	3	5	6	7	8	9
Numerical result	2.25	0.82	-0.75	0.17	0.50	-2.14

Table 12 Model 7 without composite action for a wheel-load of 200 kN

Location of measurement	Stress [N/mm^2]					
	10	11	12	13	14	15
Numerical result	6.6	6.6	8.1	7.6	36.5	47.5

steel plate are significantly reduced following from the lacking composite action. Compared with the stiffened deck of Test Model 3 the vertical deformations below the wheel-load increased by a factor of roughly 1.5, while those of the transverse rib (together with the stresses there) remained nearly the same. This behaviour altogether indicates that the composite action is much more effective for the local load-distribution in case of the unstiffened deck than for the stiffened deck.

3.4. Summary

The investigation of the composite deck construction combining a RC plate with an unstiffened deck plate both spanning over the distance between the transverse ribs showed a similarly stiff local load-carrying behaviour as for the stiffened deck. This is true in particular for the case that rigid composite action is achieved by shear studs plus bonding between concrete plate and steel plate. This bonding effect was intact up to a load-level which was 3.5 times as high as the maximum wheel-load according to EUROCODE.

When the bonding effect had been overcome, the structure lost some of its stiffness due to the ductility of the shear studs. However, the test results in this case may be considered as rather conservative, since in the real structure the locally highly stressed concrete plate is surrounded by the rest of the plate, activating an additional restraint against differential movements between concrete plate and steel deck.

Altogether, it may be summarized that also composite deck constructions built up by a unstiffened deck plate and a RC plate form an effective structure for a traffic deck.

4. Conclusions

Investigations, both experimentally and numerically, were made for clarifying the load-carrying behaviour of composite deck constructions built up by a RC plate and a steel deck, which may be a stiffened orthotropic plate or an unstiffened plate. The connection in both cases was made by shear studs.

The outcome of the present study was that the given construction provided an effective load distribution over a local area under concentrated wheel-loads and that also under dynamic loading over 3 million load cycles this beneficial behaviour could be maintained. Accordingly the structure was appropriate also for use as traffic deck.

Secondly, the study also verified that design of such structures may be carried out on the basis of numerical analyses, since good agreement could be achieved between tests and numerical simulations.

Although a number of different models have been studied, these were related to the specific conditions of a given steel deck. Accordingly, the above conclusions are bound to the given range of parameters.

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