III a 2

Tests with welded light-weight deck structures

Versuche mit geschweissten Leichtfahrbahn-Konstruktionen

Ensaios de estruturas ligeiras de tabuleiros soldados

Essais sur charpentes légères de tabliers soudés

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A. *Description of the structures tested*

Preliminarily to the construction of a new highway-bridge over the Danube in Budapest, elaborate large-scale tests were carried out with various types of welded bridge floor structures. The test elements were prepared in natural size with 9,30 m length and 7,0 m width and comprised 5 different types (Fig. 1).

Test floor-system n° I. was composed of cylindrical steel shell segments, 5 mm thick, supported underneath by welded T beams at 1380 mm intervals and stiffened at the top with quality concrete filling layer, 3 cm thick over the crown and 10 cm at the spandrels, the cooperation of which was secured by simple oblique dowels of flat-steel, placed at 50 cm intervals and welded along two rows on the extrados of the shell. In addition, a steel wire-mesh of 1 mm ϕ wires and 4 cm spacing was laid just below the top-surface of this concrete filling. Thereupon was laid the road surfacing, consisting of a 1 cm thick bituminous insulating layer, 4 cm protective layer of concrete and 5 cm asphalt coating on top, which, actually, was the same for all systems.

N° II. was of the same type, differing only in the spacing and form of the upper stiffening diaphragms of the shell plate.

The floor-system n° III. was composed of a flat steel-sheet (orthotropic-plate) 5 mm thick, welded on underlying steel-joists N° 24, also placed at 1380 mm intervals. The plate was stiffened from underneath by 200 × 8 mm diaphragms and a certain cooperation was secured at the top by welded sinusoidal standard road-surface reinforcing mat of flat bars (15 × 3 mm).

Test deck Nº I.

Fig. 1. Cross-section of the tested light-weight deck-structures

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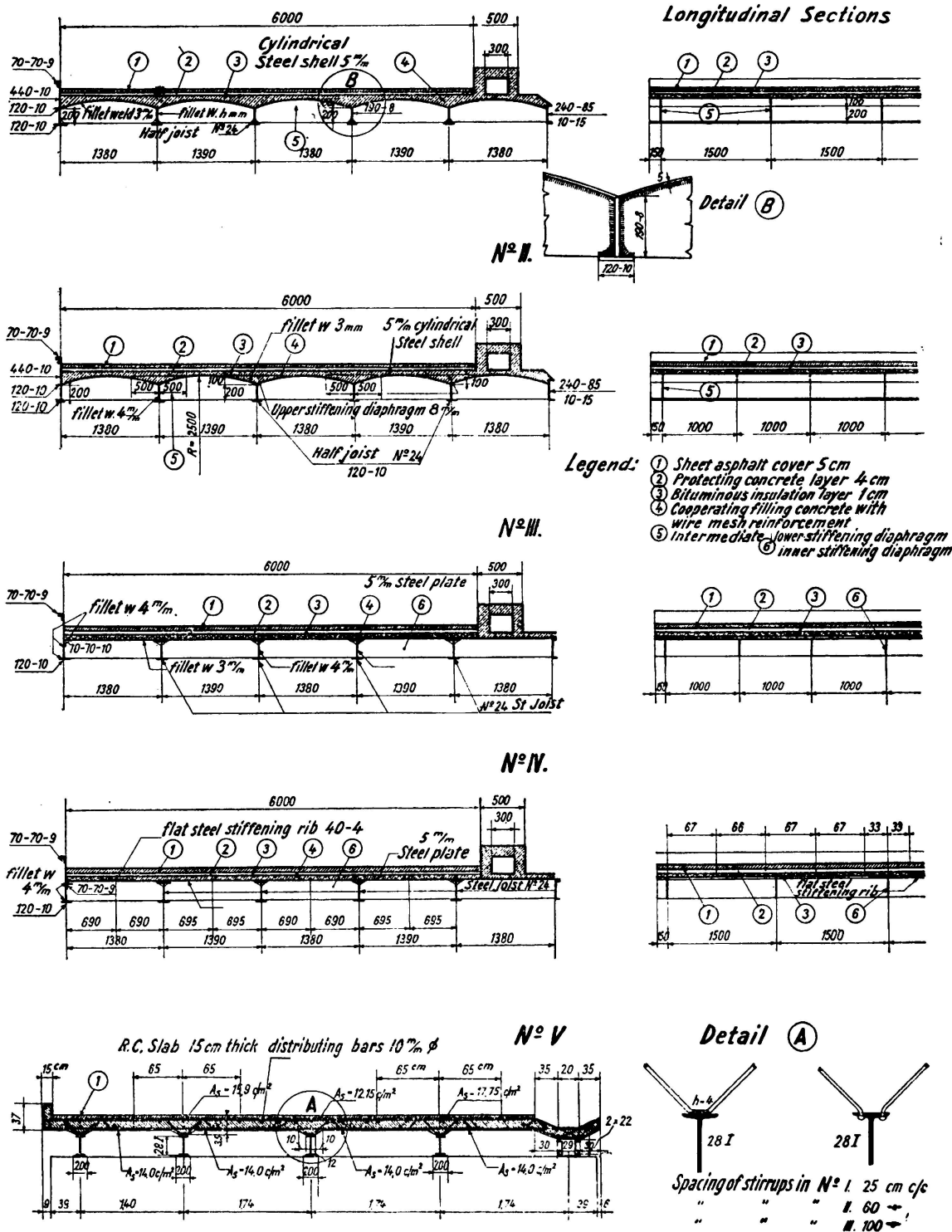
N° IV. was also of the flat-steel sheet type, differing from N° III again in the spacing and type of stiffening elements. Here the upper system consisted of 40 × 4 flat bars welded in a net of 67 × 140 cm.

N° V. was of the normal composite girder type, consisting of N° 28 steel joists cooperating with the upper 15 cm thick slab.

Compared to the the usual deck constructions the reinforced concrete tests-floors N°ˢ I-IV. showed a reduction in deadweight of about 30-40 % with the same quantity of steel (varying between 230-280 kg/cm² and 80-98,2 kg/m² resp.) while the composite structure N° V. showed a reduction of steel of 30-40 % while keeping the same dead-weight figures
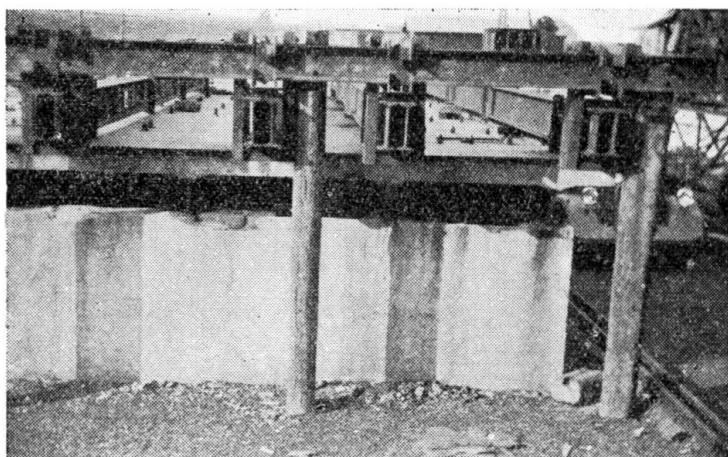
Fig. 2. View of the test-installation with supporting reinforced concrete piers and cantilever loading arms

(54 kg/m² and 420 kg/m²). The steel structure of the test pieces was delivered after fabrication to the test-field and placed on four reinforced concrete supporting columns spaced at 3,0 m intervals (in conformity with the distribution of cross-girders on the bridge).

Statical test loading was carried out through cantilever arms extending at the center of each span over the whole width of the test piece, and was effected in five variations (Fig. 2).

Dynamic loading was carried out by a vibrating machine of the *Losenhausen-type,* weighing 1140 kg and having a centrifugal force of 2000 kg. The static weight of this apparatus was increased to 3500 kg by means of counterweights which corresponded practically to the maximum wheel load.

B. *Test results*

1) *Stresses in the stringers*

Fig. 3. shows the measured tensile stresses (and their mean values) at the lower-fiber of the longitudinal stringer elements in cross-section «C» under the loading axis «α» and «α + β». It was found that these

values corresponded to the calculated values only when a *perfect coope-ration* was assumed between the steel shell-plates and the filling concrete layers. It was also found, that the maximum values did not reach the permissible stress-limit of 1400 kg/cm², but a considerable safety margin
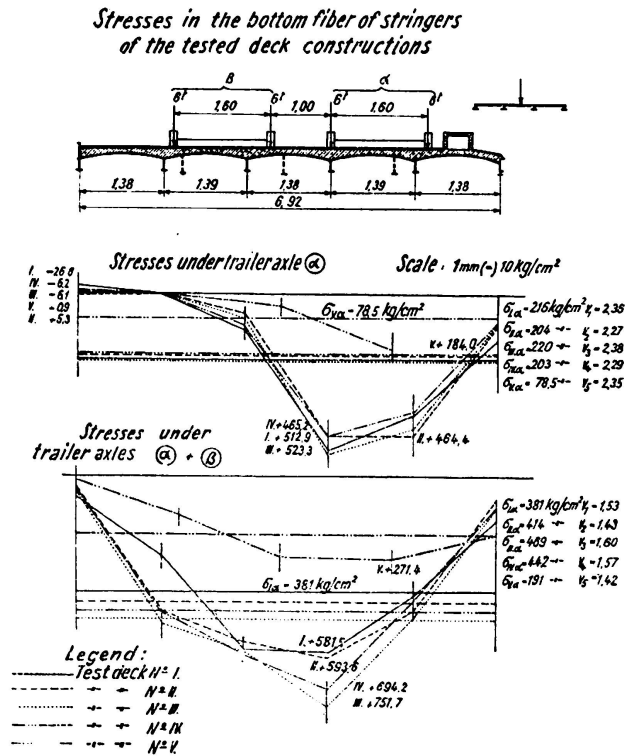
FIG. 3. Measured stresses under static loading
with two 12ᵗ axes

was left. In all cases, even if the impact factor of *1,5* was considered, the

ratio of $\dfrac{\sigma_{perm}}{\sigma_{max}} = \nu$ was well beyond unity and varied with the different

types as follows:

$$N^\circ\ I. \qquad \sigma_{max} = 1033\ \text{kg/cm}^2;\quad \nu = \frac{1400}{\sigma_{max}} = 1{,}36$$

$$N^\circ\ II. \qquad \sigma_{max} = 1055\ \text{kg/cm}^2;\quad \nu = \frac{1400}{\sigma_{max}} = 1{,}33$$

$$N^\circ\ III. \qquad \sigma_{max} = 1336\ \text{kg/cm}^2;\quad \nu = \frac{1400}{\sigma_{max}} = 1{,}05$$

$$N^\circ\ IV. \qquad \sigma_{max} = 1246\ \text{kg/cm}^2;\quad \nu = \frac{1400}{\sigma_{max}} = 1{,}12$$

$$N^\circ\ V. \qquad \sigma_{max} = 503\ \text{kg/cm}^2;\quad \nu = \frac{1400}{\sigma_{max}} = 2{,}78$$

These figures show the absolute superiority of the composite-deck N° V. and the relative superiority of the cylindrical-shell types N° I. and II.

2) Stresses in the cylindrical or flat plates.

This steel sheeting is subject to composite stresses longitudinally, acting as compression flanges of the stringers and transversally by their direct load carrying part between the stringers. The longitudinal component is largest at the stringer axis itself, whereas the transversal component is preponderant at the center of each bay. Therefore the cross-
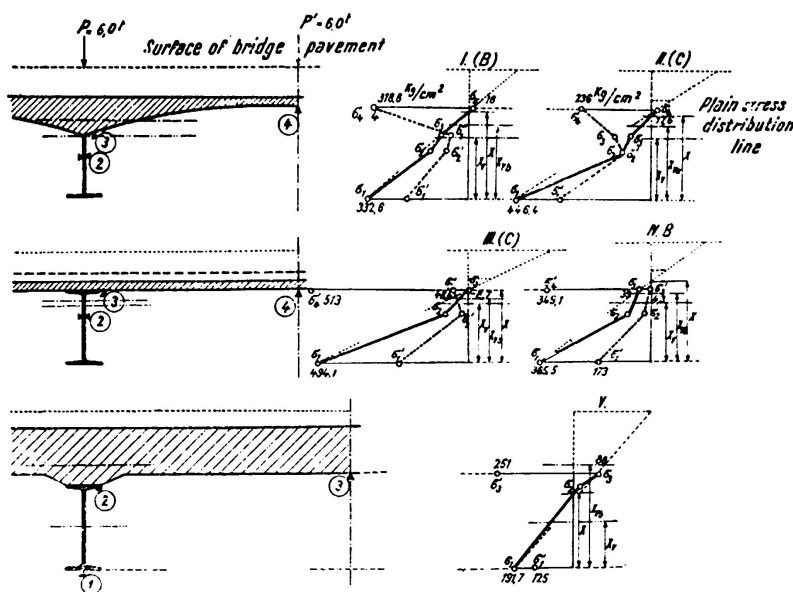
Fig. 4. Measured distribution of stresses in the cross sections

-section may be regarded as uniform only when the stiffening of the sheeting is sufficient to ensure a perfect cooperation between stringer and sheeting, by means of the diaphragms. Fig. 4. shows the stress distribution flow in one cross section. The location of tensometers is shown and it is seen, that the measured values do not form a continuous line, a decisive break being visible at the junction plane of the sheeting. Where the lower stiffening diaphragms are stronger (N° I. and N° III.) this break is, of course, smaller. The location of the load also makes a great difference in the stress distribution. This is shown by the dotted lines, which represent the case when the wheel load is acting at the center of the span. The most regular stress distribution line was obtained with the ordinary composite-girder type where this is due to the large cross-sectional rigidity of the reinforced concrete slab.

The stress diagrams also indicate, that *compression was carried almost exclusively by the filling concrete* or by the reinforced concrete slab, thus giving an excellent *proof of cooperation.*

The numerical values of the measured stresses were well under the calculated ones, whatever the theory they were compared to (Marcus etc.).

The ratios of $\nu = \dfrac{\sigma_{perm}}{\sigma_{max}}$ of the tested types were the following:

$$
\begin{array}{llll}
\text{N}^\circ \text{ I.} & \text{Test piece} & \nu = 4,70 & (3,04) \\
\text{N}^\circ \text{ II.} & \text{»} \quad \text{»} & \nu = 3,15 & (2,04) \\
\text{N}^\circ \text{ III.} & \text{»} \quad \text{»} & \nu = 1,80 & (1,80) \\
\text{N}^\circ \text{ IV.} & \text{»} \quad \text{»} & \nu = 2,70 & (2,20) \\
\text{N}^\circ \text{ V.} & \text{»} \quad \text{»} & \nu = 2,75 & \\
\end{array}
$$

These results clearly show, that the lower diaphragms are more effective (N° I. and IV.). An effectively cooperating upper concrete filling layer, 3 cm thick was quite sufficient to prevent any kind of buckling. Furthermore the superiority of cylindrical shells (Test piece N° I.) appears clearly over flat steel sheets. (N° III. and IV.). (This is naturally partly due to the greater structural depth; the reduced «ν» values referring to a reduced uniform depth are therefore shown in dashes).

3) Tests of composite action

In order to prove the value of composite action all the test-constructions finally underwent a very serious dynamic-test loading. The Losenhausen-type oscillator was placed and anchored at the center of the middle span and the test constructions were subjected in turn, to their own oscillation frequency, to 6 million loading cycles. The stresses and deflections of all stringers were previously measured under this static loading, checked constantly during the test and finally controlled after 6 million loading cycles. It was seen that, with no exception, neither the stresses nor the deflections increased.

Then the span of the test constructions was increased to 6 m and all of them were subjected again to 6 million additional loading cycles. The ensuing control-measurement showed a general increase in the deflections amounting up to 7-26 %, although, even then no failure took place. This indicated however a certain slackening of the bond between steel and concrete. (The greatest deviation was shown by test-deck N° II. which indicated that the upper stiffening diaphragms were not equal to the lower ones). Finally all test-constructions failed after a few thousand loading cycles when oscillated again over a 9,0 m span.

It must also be pointed out, that the favourable fatigue results were partly due to the favourable location of welds which were at about the level of the neutral axis.

4) Transversal load distribution

Test results are shown on Fig. 5. The dotted lines represent the calculated values computed after Leonhardt, and the full lines correspond to the measured values. It is quite clear that the peak values of these load distribution influence lines are smaller than the calculated ones, and

no negative values were measured in the neighbouring spans. The loss in the area is however equalised either by the larger area of the neighbouring spans or by the larger load transmission capacity of the structures.

The effect of the cooperating slab-width and of the rigidity of the stiffening diaphragms is considerable. The second diagram of Fig. 5 clearly indicates the difference of the load distribution influence lines, when no diaphragm (dotted line) and when a simple diaphragm (190×8 web
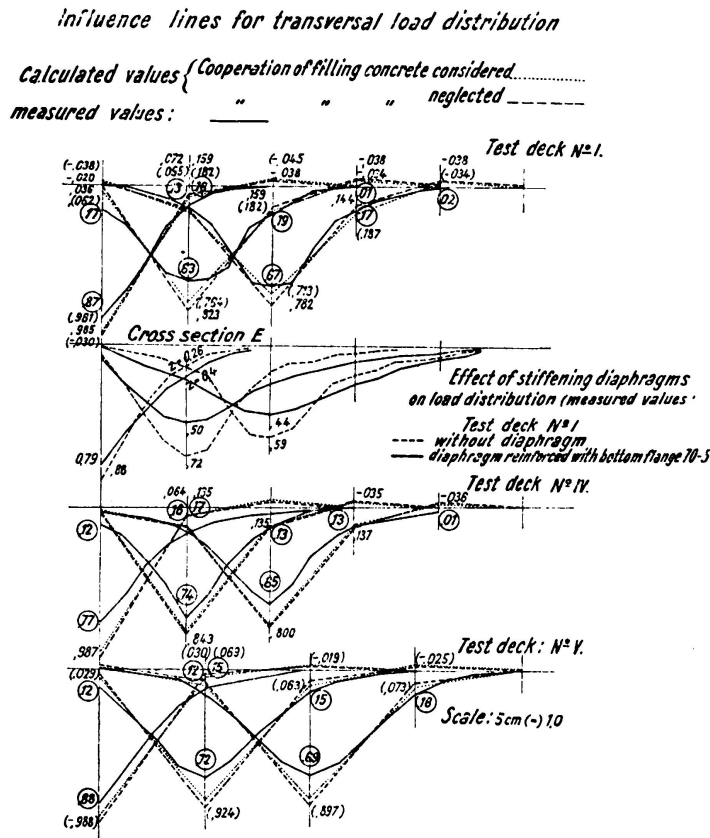

Fig. 5. Influence lines for transverse load
distribution

70 × 8 flange, full line was applied. It also appeared from the tests, that the increase in rigidity of the transversal diaphragms is not in direct ratio with the load distributing effect, but beyond a certain limit $\left(\text{if} \quad z = \dfrac{I_a}{I} \cdot \dfrac{l^3}{2\,a^3} = 0,5 \right)$ it is not economical.

The *cooperating slab-width* also differs from the usually calculated strip-width. The author has stated, that the working surface of the slab under a point load will take the shape of a rhomb, extending in longitudinal direction to the nearest cross girders and in transversal direction over the two neighbouring spans. (Fig. 6). For a section at a distance x from the point-load, this width may be expressed as $s = l \left(1 - \dfrac{4\,a}{x} \right)$ until $x \leq 2\,a$ and $s' = \dfrac{l \cdot a}{x}$ when $x \geq 2\,a$.

The results of the test may finally summarised as follows:

1) The application of well bonded and reinforced filling concrete, laid over cylindrical steel shell-plates or flat-steel sheeting, even in a minimum thickness of 3-5 cm-s, is very advantageous and raises considerably its bearing capacity.

2) The composite action between steel sheeting and concrete may be ensured by the simplest means (dowels and wire mesh) and an additional 2-3 cm thick layer of concrete may suffice as wearing surface, when the steel is noncorrosive and is alloyed with 0,3-0,5 % Cu.
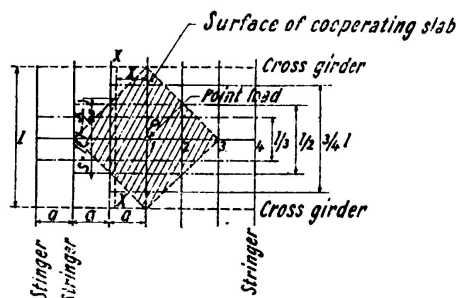

FIG. 6. Proposed shape of cooperating slab-width

3) The rigidity of the whole deck-construction may be very effectively raised by simple diaphragms welded underneath the sheeting. These are more effective than those welded on the top.

4) The use of cylindrical shell-plates seems to yield certain advantages as to the bigger rigidity, less weight and favourable location of welded joints but the transversal load transmission capacity is smaller, than that of the flat-steel sheeting (orthotropic-plate).

5) The cooperating slab-width may be better computed by the advised approximation, than with the usual prescriptions, which did not correspond to the test measurement.

6) The load transmitting action must not be extended beyond the two neighbouring spans on each side. The trend to increase this by raising the rigidity of the transversal diaphragms is economical only to a certain extent, the limit being such that *Leonhardt's* z value should not usually exceed 0,5 or as an ultimate maximum: 1,0.

SUMMARY

Full size tests were carried out with five types of light-weight deck-structures. Stringers and slab of all five types operated as composite structures. Altogether two welded cylindrical shell-plate, two welded flat-shell-plate (orthotropic) and one current composite structures were tested.

The measured stress and deflection values were compared with theoretical values (Leonhardt, Marcus) and it was shown that cooperation between the composite elements is more favourable than expected; this cooperation can be ensured, even between the steel plate and filling concrete layer, by the simplest means. The favourable location of welds and the effective bond of composite elements has also proved successful against fatigue action.

The stress- deflection diagrams and the comparative figures give ample information on the test results.

ZUSAMMENFASSUNG

Grossversuche mit fünf verschiedenartigen Leichtfahrbahnkonstruktionen. Die Versuchskonstruktionen wurden statischen Radlasten und dann dynamischen Schwingerlasten unterworfen.

Die Längsträger und Deckplatte wurden bei allen Typen als Verbundkonstruktionen ausgebildet.

Es wurden insgesamt 2 geschweisste Buckelblechdecken, 2 geschweisste Flachblechdecken und 1 Verbundkonstruktion geprüft.

Die gemessenen Spannungs- und Durchbiegungswerte wurden mit den Angaben der annähernden Berechnungsergebnisse (Leonhardt, Marcus) verglichen, und es hat sich ergeben, dass die Lastverteilung zwischen den einzelnen Elementen in der Tat viel einheitlicher und günstiger ist. Die Verbundwirkung selbst zwischen Buckelblechen oder Flachblechen und Ausfüllbeton kann mit den einfachsten Mitteln und schon bei ganz geringen Beton-Abmessungen wirksam gesichert werden. Die günstige Anordnung der Schweissstellen und der wirksame Verbund zwischen Stahlplatte und Ausfüllbeton bietet ein Gewähr gegen Ermüdungserscheinungen.

Die Spannungs- und Durchbiegungsdiagramme und vergleichenden Zahlentafeln geben eine klare Übersicht über die Versuchsergebnisse.

RESUMO

Efectuaram-se ensaios à escala natural em cinco tipos diferentes de estruturas de tabuleiros em que as vigas e a laje trabalhavam em conjunto. Ensaiaram-se ao todo duas estruturas com chapas cilíndricas soldadas, duas com chapas planas (ortotrópicas) e uma de construção mixta corrente.

Os valores medidos das tensões e das flechas foram comparados com os valores teóricos (Leonhardt, Marcus) e verificou-se que a cooperação dos elementos mixtos é mais favorável do que é normalmente admitido; esta cooperação pode ser obtida, mesmo entre a chapa e a camada de betão de enchimento, por meios muito simples. A localização favorável das soldaduras e a ligação efectiva entre os diversos componentes também provaram ser eficazes na resistência à fadiga.

Os diagramas tensão-deformação e os valores comparativos dão uma ideia clara dos resultados dos ensaios.

RÉSUMÉ

Des essais en vraie grandeur ont été effectués sur cinq types différents de structures de tablier où les poutres et la dalle travaillaient ensemble. Ces essais ont porté sur deux structures comprenant des tôles cylindriques soudées, deux comprenant des tôles planes (orthotropiques) et une en construction mixte courante.

Les valeurs mesurées des contraintes et des flèches ont été comparées aux valeurs théoriques (Leonhardt, Marcus) et il a été constaté que la

coopération des éléments mixtes est plus favorable qu'il n'est générale-
ment admis; cette coopération peut être obtenue, même entre la tôle et
la couche de béton de remplissage, par des moyens très simples. La dis-
position favorable des soudures et la liaison effective entre les divers
éléments se sont également révélés efficaces contre les effets de la fatigue.

Les diagrammes contraintes-déformations et les valeurs comparées
permettent de se faire une idée claire des résultats des essais.