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Autor: Arda, T. Seno / Yorgun, Cavidan

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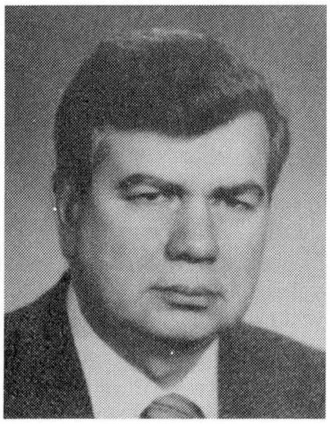
Deflection of Composite Slabs Connected with Welded Lattice

Flèches des dalles mixtes avec treillis soudé comme connecteur

Durchbiegung von Verbunddecken mit angeschweissten Stahlmatten

T. Seno ARDA
Prof. Dr.
Istanbul Technical Univ.
Istanbul, Turkey

Cavidan YORGUN
Dr. Eng.
Istanbul Technical Univ.
Istanbul, Turkey



T. Seno Arda, born 1940, received his M.Sc. degree at ITU, the Istanbul Technical University. Research assistant for two years at Liège University Belgium, where he prepared his doctorate thesis. He started his academic career when he returned to ITU, where he is now coordinator of the Steel Structures Group.

Cavidan Yorgun, born 1960, received her M.Sc. and Ph.D. degrees at the Istanbul Technical University where she is now research assistant in the Steel Structures Group.

SUMMARY

This paper presents the part of the results related to the serviceability limit states of an experimental research investigation on composite slabs in which the connection between the concrete and the profiled steel sheeting is obtained by a reinforcing lattice welded on the steel sheeting. This type of connection is quite practical and prevents bond failure. The paper is concluded with some design specifications and an interpretation of results.

RESUME

Cet article présente la partie des résultats aux états limites de service, d'une recherche expérimentale ayant pour sujet les dalles de plancher mixtes formées de béton coulé sur des tôles d'acier profilées, la connection entre les deux matériaux étant obtenue par un treillis soudé sur la tôle. Ce type de connection est assez pratique et empêche toute rupture par adhérence. L'article se termine par l'interprétation des résultats et par quelques considérations pour le projet.

ZUSAMMENFASSUNG

In dieser Arbeit werden Versuchsergebnisse zu den Gebrauchsgrenzlasten aus Betondecke und Stahltrapezprofil bestehenden Verbunddecken angegeben, an denen die Verbundwirkung durch Stahltrapezprofil angeschweisste Betonstahlmatten gesichert wurde. Diese Verbindungsart ist ziemlich praktisch und verhindert einen Verbundbruch. Die Berechnungsverfahren und Folgerungen beschliessen die Arbeit.

1. INTRODUCTION

In floor construction of steel skeleton buildings, there is an increased use of composite slabs with profiled steel sheeting.

In order to permit the steel deck to serve as tensile reinforcement and act compositely with the concrete, a means for positive mechanical interlock is needed. This mechanical interlocking is in most cases totally provided by the mechanical shear transferring devices [1, 2]. These are:
- Indentations or/and embossments rolled in to the deck,
- Study connectors welded on supports,
- Reinforcing steel lattice welded on the steel sheeting.

In Turkish market, steel decks' surfaces are commonly smooth. For this reason, the connection between the concrete and the profiled steel sheeting is oftenly obtained by a reinforcing lattice welded on the sheeting. This type of connection is practical and prevents generally an adherence rupture. This welded lattice, on the other hand, is also functioning as a load distributing reinforcement in transverse direction. Since the quality of connection provided by such a connection was not exactly known, it was decided to test it experimentally.

2. DESCRIPTION OF TEST SLABS

In this experimental study; a preliminary series of 6 tests and a main series of 9 tests are carried out in the positive bending moment zone to investigate the ultimate and serviceability limit states [3]. The profiled steel sheetings used in test specimens are galvanized and show three different thicknesses (t=0.75mm, t=1.00mm, t=1.20mm), same width (b=860mm) and depth (d_a=27.5mm). The typical cross section of steel sheeting is shown in (Fig. 1). The concrete was casted in such a way that the total depth of slab is nominally equal to d_t=10cm. A summary of the main series' specimens properties is given in Table 1.

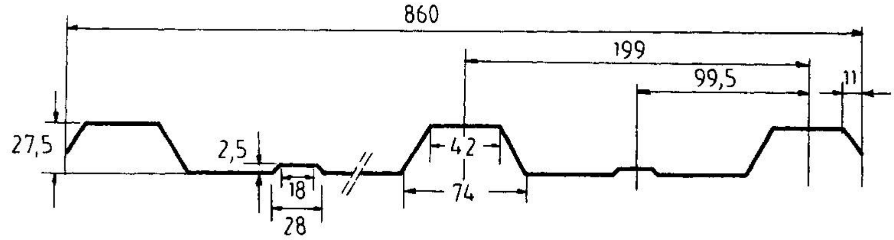


Fig. 1 Typical cross-section of the steel sheeting

Slab Element	Shear Connection Lattice	Depth of Slab Element (Measured)	Compressive Strength of the Concrete	Yield Strength of the Steel Sheeting
	mm	cm	daN/cm^2	daN/cm^2
3.A-0.75 3.B-0.75 3.C-0.75	Ø4.5/150 (150.250.4,5.4,5)	10.5 10 10	300	2850
4.A-1.00 4.B-1.00 4.C-1.00	Ø4.5/150 (150.250.4,5.4,5)	11 9.5 10	450	2400
5.A-1.20 5.B-1.20 5.C-1.20	Ø5/150 (150.250.5,0.5,0)	10 10.5 10	350	2850

Table 1 Summary of the main series' specimens properties

3. TEST PROCEDURE

The test specimens with a span of 300 cm, were simply supported at both ends and the test frame was designed to apply two concentrated loads at the one thirds of their span lengths (Fig. 2), such a loading providing a bending moment diagram similar to the uniform loading.

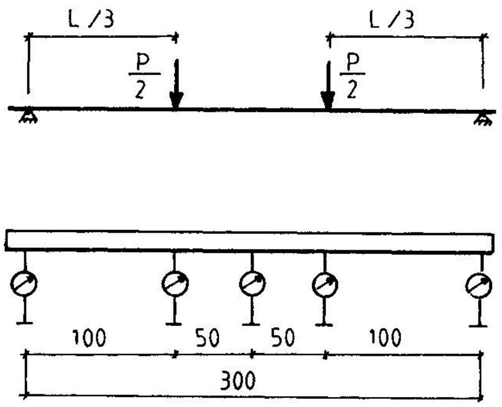


Fig. 2 Typical arrangement for testing slab elements

The load P applied to the test specimens was static in character and was increased in steps starting from zero up to the level where the failure mechanism was observed. At each increment of the loading, the deflections at the mid-span (L/2) and at one thirds of the span (L/3) along with the strains at various locations of the steel sheeting and the concrete were both measured.

The experiments were performed at the Structural Laboratories of Istanbul Technical University.

The primary test series are conducted to determine a suitable shear connection calculation which could prevent an adherence rupture.

The shearing force between the steel sheeting and the concrete can be assumed equal to the tensile capacity N_s of the steel sheeting.

$$N_s = A_s f_y$$

Where A_s is the cross-sectional area and f_y is the yield stress of the steel sheeting. It is easy to compute

$$A_{sl} > \frac{N_s}{2 n_H m_H \tau_{all}}$$

Where n_H is the number of ribs of the steel sheeting, m_H is the number of transverse bars of the lattice along one shear span, A_{sl} is the cross-sectional area of this bar and τ_{all} is allowable shear stress of this bar.

The reinforcing lattice designed using this method, shear-bond failure has never been observed in the test specimens before the flexural failure.

Ultimate limit state loads of the test specimens and their serviceability limit states were both investigated. This paper presents the part of the results related to the serviceability limit states of an experimental research.

4. METHODS OF ANALYSIS OF THE DEFLECTION

In calculating the theoretical deflections for a comparison with the experimental deflections, the moment of inertia were obtained seperately for the cracked and uncracked cross-section. These two values and also their average as advised in EC4 [4] and ASCE [5], were used in calculations. In another approach, only the concrete located upon the steel sheeting profile was taken into account in the calculation of the moment of inertia.

In transforming the concrete cross-section to an equivalent steel cross-section in order to calculate the moment of inertia, the width of the concrete part was used divided with 2n, in which $n = E_s/E_c$ is the ratio of the modulus of elasticity of steel to concrete, this to take into account the creep of concrete.

The accordance, between the experimental an the computed theoretical deflections is remarkable with respect to two methods of calculations in which either the average moment of inertia of cracked and uncracked sections is used or only the moment of inertia of the concrete located upon the steel deck is taken into account.

From the practical point of view, these results are important only in between the two following application limits:

- The design load P^i_{ut} obtained by dividing the theoretical load P_{ut} with a safety factor η (1.7 in Turkish Standard TS. 4561) [6]
- Deflection limit load P_{us} (L/300 in Turkish Standard TS. 648) [7]

Table 2 shows measured deflections and computed deflections, at the mid-span, based on two different methods, for each slab element, when loaded to the design load P^i_{ut} in which theoretical deflection δ_{t1} is computed using a simple average of cracked and uncracked sections and theoretical deflection δ_{t2} is computed using the concrete located upon the steel sheeting.

Slab Element	P_{us}	P^i_{ut}	δ_e	δ_{t1}	δ_{t2}	δ_e/δ_{t1}	δ_e/δ_{t1}	P_{us}/P^i_{ut}
	daN	daN	mm	mm	mm	....	....	....
2.A-0.75	1950	1841	9.20	9.23	10.44	0.997	0.881	1.060
2.B-1.00	2250	2098	8.30	9.35	10.47	0.888	0.793	1.072
2.C-1.20	2300	3149	14.90	12.78	14.19	1.166	1.050	0.730
3.A-0.75	2100	1941	8.95	8.79	9.77	1.018	0.916	1.082
3.B-0.75	1750	1824	10.70	9.44	10.66	1.133	1.004	0.959
3.C-0.75	1850	1824	9.60	9.44	10.66	1.017	0.901	1.014
4.A-1.00	3100	2407	5.65	7.87	8.63	0.718	0.655	1.288
4.B-1.00	2340	1988	7.65	9.71	11.06	0.788	0.692	1.177
4.C-1.00	2550	2128	7.00	9.04	10.17	0.774	0.688	1.198
5.A-1.20	2300	3149	15.10	12.78	14.19	1.182	1.064	0.730
5.B-1.20	2460	3359	14.50	11.94	13.14	1.214	1.104	0.732
5.C-1.20	2300	3149	15.60	12.78	14.19	1.221	1.100	0.730

Table 2 Measured and computed deflections at P^i_{ut}

The typical load-deflection behavior for the test specimen (5.A-1.20) is shown as an example in (Fig. 3).

As in these the limits described above, the two methods described above give similar results, the second one which is simpler is concluded to be more appropriate for practical purposes.

One of the aims of this study was to observe either the ultimate limit state or the serviceability limit state was more influent. From the three different thicknesses of the folded steel sheetings used in tests, only the thickest one (t=1.20mm), resulted in ultimate design load greater than the serviceability limit state load. In the test specimens with the two other thicknesses: thin (t=0.75mm) and medium (t=1.00mm), the result in contrary.

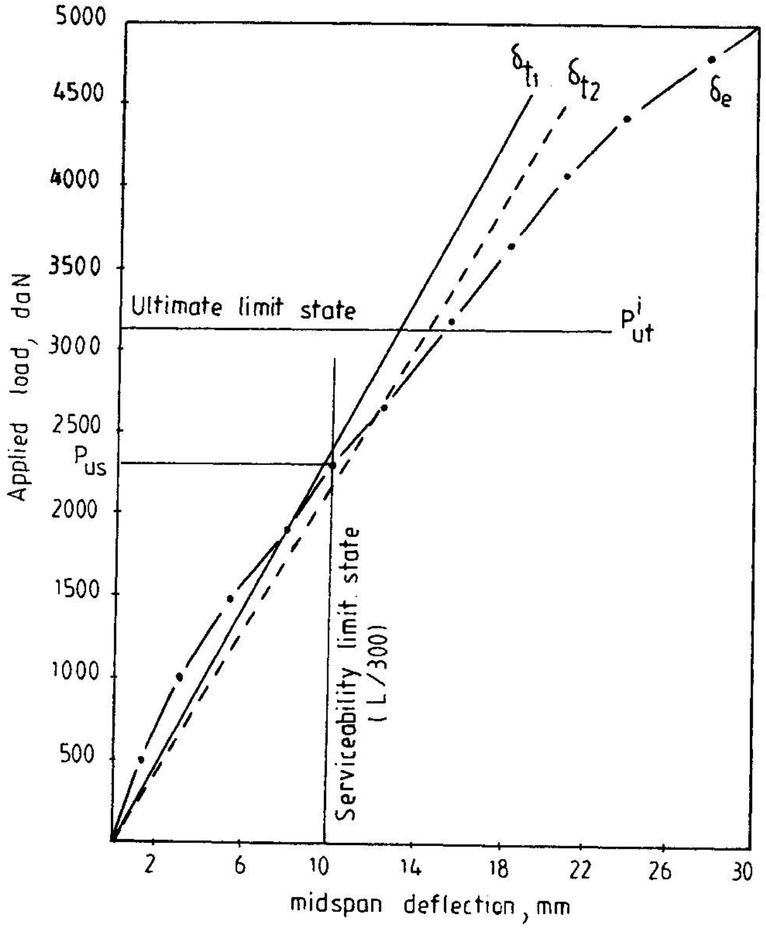


Fig. 3 Load-deflection curves for a test specimen

5. CONCLUSIONS

The accordance between the experimental and the theoretical deflections is less remarkable compared to the results reached in ultimate loads values. However, the best correlation is obtained with two methods of calculations in which either the average moment of inertia of cracked and uncracked sections, or only the moment of inertia of the concrete located upon the steel deck are used. The second approach which is simpler, is judged to be more appropriate for practical purposes.

The general evaluation of the test results show that, for the cases where, t/d ratio (thickness of the steel sheeting/nominal depth of composite slab) is smaller than, or equal to 1 percent, the ultimate limit state is more significant than the serviceability limit state.

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