COMPOSITE SLAB BEHAVIOR AND STRENGTH ANALYSIS UNDER STATIC AND DYNAMIC LOADS

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ABSTRACT

Steel-framed buildings are typically constructed using steel-deck-reinforced concrete floor slabs. The in-plane (or diaphragm) strength and stiffness of the floor system are frequently utilized in the lateral load-resisting system design. This paper presents the results of an experimental research program in which four full-size composite diaphragms were vertically loaded to the limit state, under static or dynamic loads. Two test specimens were provided with longitudinal steel-deck ribs, and the other two specimens with cross steel-deck ribs. Typical composite diaphragm limit states are described, and the controlling limit state for each of the full-size tests is indicated. The interaction effects between the reinforced concrete slab and the steel girder on the composite slab strength and stiffness were mainly studied.

Keywords: composite slabs; shear connectors; steel deck; reinforced concrete slab; steel girder

REZUMAT

Clădirile pe sisteme de cadre din oţel sunt de obicei construite utilizând plăci din beton armat pe tablă cutată. Rezistenţa şi rigiditatea sistemului de planşeu sunt utilizate frecvent în proiectarea sistemelor rezistente la acţiuni laterale. Această lucrare prezintă rezultatele unui program de cercetare în care patru diafragme compozite în mărime naturală au fost încărcate vertical până la starea limită ultimă, la încărcări statice şi dinamice. Sunt descrise stări limită ultime corespunzătoare diafragmelor compozite. Au fost în principal studiate efectele de interacţiune dintre placa de beton şi grinda de oţel asupra rigidităţii şi rezistenţei plăcii compozite.

Cuvinte cheie: planşee compozite; conectori pentru forfecare; planşeu de oţel, placă de beton armat, grindă de oţel

1. INTRODUCTION

1.1. General

The principle of composite structures is to use two or more materials in the same structural element, aiming that each material is optimal applied depending on its mechanical capabilities.

This system is applied constructively to achieve composite slabs (plates as tensioned reinforcement and framework plus concrete slab and metallic beams), columns (metallic profiles embedded in concrete or concrete-filled tubes) and composite beams for frame structures or bridges.

2. RESEARCH PROGRAM AND USED SPECIMENS

The experimental research has used four types of composite panels (M1, M2, M3 and M4) having the plates ribs differently arranged (Fig. 1.). M3 and M4 models ordered additional reinforcement connectors connecting beams-plate.

The program included experiments on static push tests (M1 and M2), and physical static and dynamic tests (M3 and M4). The slippery bearing capacity for a joining connector was established on eight push-test specimens grouped in two series (E1 and E2) - Fig. 2. Each of the two specimens of each series had additional reinforcement and Nelson connectors. The analytical assessments have used three different methods of calculating the strength and stiffness of the plate specimens, depending on specific conditions. The same methods were used for final assessment of the connectors shear bearing capacity. Comparisons between the design procedures results are made, in order to establish the most adequate procedure for design.

3. EXPERIMENTAL TESTS AND ANALYTICAL EVALUATION

3.1. Push-test specimens

3.1.1. The phenomenon of transmitting the relative plate-beam sliding to the anchorage connector is characterized especially by the structural conditions, which are including: the material characteristics (steel, connectors, concrete), extra reinforcement connectors depending on the layout of the plate ribs, the bearing beam and the plate ribs arrangement and the metal profile direction.

3.1.2. The push-test specimens of the E2 series don't give any signals for the increase of taking over the relative plate-beam sliding with additional reinforcement of the connectors, due to the manner of disposal of ribs perpendicular to the profile.

3.1.3. Regardless of the direction of the ribs arrangement, the ability of taking over the relative sliding for a connector is around 9000 daN.

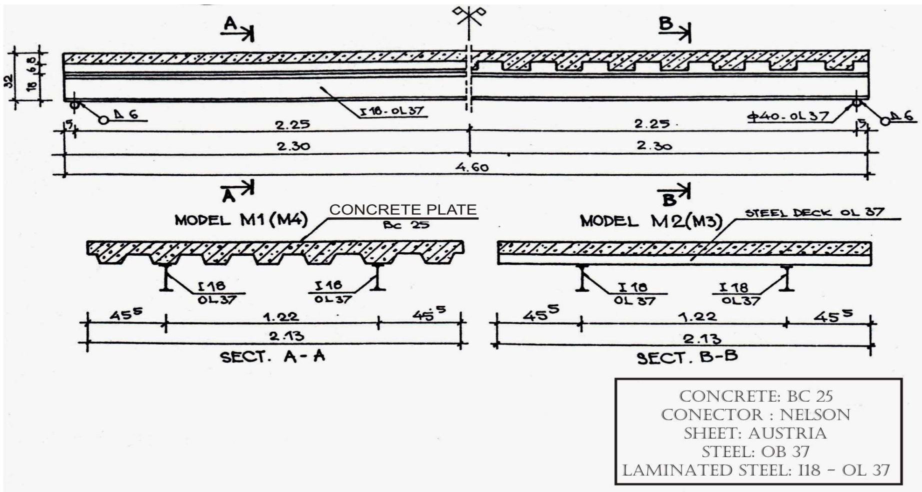

Fig. 1. Models of composite slab steel-deck type, view and sections

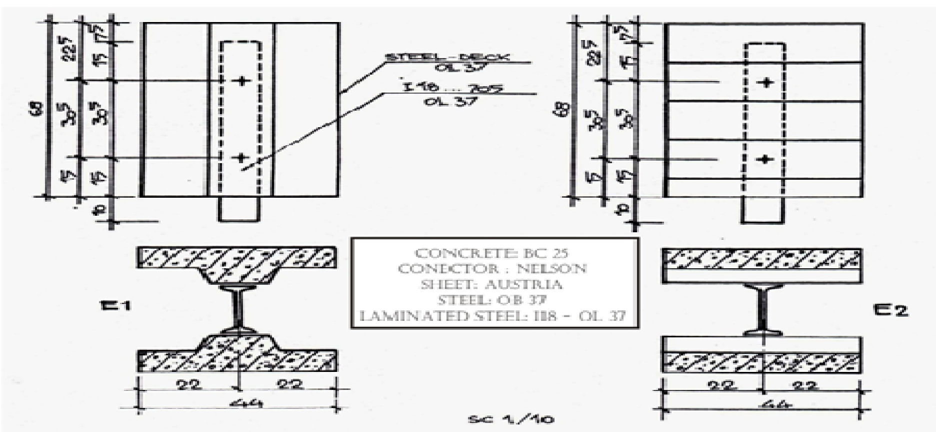

Fig. 2. Push-test specimens

An increase of 35% can be detected for the reinforced connectors located in the ribs disposed parallel to the steel beam. The value of 9000 daN / connector provides a good assessment for calculations according to (4), with maximum errors of ± 3%.

3.2. The M1-M2 plate specimens:

3.2.1. Due to the above, M1 model proves to be stiffer under vertical displacement than model M2. This is due mainly to the longitudinal ribs layout on the prop support beam's direction.

3.2.2. In the loading stage of about 80% of the corresponding to the ultimate limit states, the difference of stiffness in vertical displacement is 39% versus the model M1 instead the model M2.

3.2.3. In the ultimate limit states loading stage, the difference of stiffness in the vertical displacement of the model M1 instead the M2 is about +409.9%. This expresses a differential flexibility of composite slabs type function of the ribs layout on the supporting beams.

Conclusions:

– Ability of the plate – beam's taking over capacity of the sliding force can be increased by ca. 35% by connectors confining, only if the ribs are disposed parallel to supporting beams;

– The parallel arrangement of the ribs to the supporting beams leads to the obtaining of a coflexure (for a plate span / plate length = 0.50) lower by about 50%, on the limit state of normal exploitation, than when the ribs are disposed perpendicular on the prop laminated profiles.

3.3. The M3 model

3.3.1. Limit states of normal exploitation, according to a uniformly distributed load on the slab of about 200 daN/m are achieved for a 1577 kgf load on the semi-slab in static and 2314 in dynamic, which corresponds to an average maximum coflexure D_{max} = 0.705 mm (static) and D_{max} = 0.942 mm (dynamic). The relative maximum coflexure is $D_{rel} = D_{max} / U$ = .942 / 4500 = 0.21%, the model having an elastic behavior. No relative slab - beams sliding can be observed.

3.3.2. The last two cycles of loading - unloading in static pulse regime were done to a maximum load on semi-slab of 23 740, which takes a coflexure corresponded to a maximum average D_{max} = 49 750 mm, 3.87 times higher than allowable coflexure. At this load value, the M3 model enters in the flow stage for a load of 1146 higher than the calculation estimation. Maximum relative slab-beam sliding are reaching values of 4765 mm and are situated below the flow value $L_{c\ min}$ = 6.200 mm, with about 23%. A marginal connector, bear a sliding load of maximum around 6840 daN, 76.85% of L_c.

3.3.3. Under dynamic regime, the frequency of oscillation in the vertical translation free damped vibration remained constant for a dynamic tests 1 and 2, f = 14 Hz. The percentage of critical damping has increased from ν% = 1.31 - Exp.1 dynamic to ν = 1.88% Exp.2 dynamic with no notable degradation of the slab - beam connection during oscillation.

3.4. The M4 model:

3.4.1. Limit states of normal exploitation, according to a uniformly distributed load on the slab of about 200 daN/m are achieved for a 1713 kgf load on the semi-slab in static and 13705 in dynamic, which corresponds to an average maximum coflexure D_{max} = 0.747 mm (static) and D_{max} = 5.441 mm (dynamic). The relative maximum coflexure is $D_{rel} = D_{max} / L_0$ = 5.441 / 4500 = 0.21%, the model having an elastic behavior. No relative slab - beams sliding can be observed.

3.4.2. The last two cycles of loading - unloading in static pulse regime were done to a maximum load on semi-slab of 28810daN, which takes a coflexure corresponded to a maximum average D_{max} = 30.930 mm, 4.06 times higher than allowable coflexure. At this load value, the M4 model does not enter in the flow stage achieved by test 4 static. Maximum relative slab-beam sliding are reaching values of 2.124 mm and are situated below the flow value Lc min = 3.750 mm, with about 43%. A marginal connector bore a maximum sliding load of around 5098 daN, 56.64% of L_c.

Stage of flow (for instance, four static) to develop these phenomena:

– maximum flow load for model M4, P_c = 38 800 daN / semi-slab;

– peak coflexure achieved, D_{max} = 57 440 mm;

– maximum sliding achieved, L_{max} = 4268 mm.

3.5. The M3 - M4 models:

3.5.1. In the limit states of normal exploitation, even if some differences are reported between the loads and vertical displacement corresponding to the two models, different arrangement of the metal ribs do not give out M3 model from the actual field use. The M3 model with transverse ribs on the profile occurs much more flexible than model M4, which is explained by the different way of working of plate - concrete - connectors system. In the limit states of normal exploitation M4/M3, stiffness difference is 3% for both static load and for the dynamic as well.

3.5.2. In the ultimate limit states stage (Exp.3 – M4 and M4 Exp. 3 - M3) difference in stiffness increases to 95.20% for the model with ribs arranged longitudinally with steel “I18” beams (M4).

3.5.3. In the case of the supplementary reinforcement of the Nelson connectors, the conventional sliding beam – slab stiffness, the ultimate limit state is higher M4 model of 2.72 times than the model M3. This work reveals the different specimens behavior depending on the adopted structure.

3.6. Comparison between experimental results obtained on specimens M1, M2, M3 and M4:

3.6.1. In the state limit of normal exploitation at similar levels of action with imposed vertical load, models M1, M3 and M4 (except M2) present comparable stiffness on vertical displacement irrespective of the embedded connectors reinforcing mode or the arrangement of the ribs to the longitudinal direction of steel profiles.

3.6.2. The maximum coflexure of the bearing beam profile 118 - OL37 is recorded for the M4 model (excepting model M2) and has a value of 0.746 mm, 17 ‰ or 1 / 5900 of the span.

3.6.3. Is clear that, at ultimate limit states of normal exploitation (SLEN), a bended plate model by one direction: plate length / plate width = 4.50 / 2.11, with elastic behavior (M1, M3, M4), the layout of ribs by the bearing plates axis does not significantly affect the stiffness characteristics of the slab.

3.6.4. The final stiffness in the vertical displacement for M1 and M4 models, with ribs arranged longitudinally with the axis of the core, compared to models M2 and M3 with ribs arranged perpendicular to the bearing profiles is 2.37 times greater.

$$\frac{Rig.M1 + Rig.M4}{Rig.M2 + Rig.M3} = \frac{1090 + 675}{266 + 477} = 2.37 \quad (1)$$

3.6.5. The decrease of the ULS stiffness as compared to SLS is on average in the same ratio, 1.75 times:

$$\frac{0,543(M1) + 0,294(M4)}{0,265(M2) + 0,213(M3)} = 1.75 \qquad (2)$$

3.6.5. At ultimate limit states, the plates models with longitudinal arranged ribs to the beam 118 has a final relative sliding stiffness 1.53 times higher than the similar models with ribs perpendicular to the profiles.

3.6.6. Average percentage of maximum beam – slab sliding from the capable sliding is between the values:

– Model M1, M4, with parallel ribs: Max sliding / capable sliding = 0.628

– Model M2, M3, with perpendicular ribs: max sliding / capable sliding = 0.646 are comparable values.

3.7. Comparative analytical – experimental analysis

3.7.1 The "A" & "B" methods for analysis (2):

– The design load for plastic hinge occurrence in the middle of the I18 steel beam profile, computed with the theoretical values of compressive strength of slab concrete shows the deviations from the obtained similar results by adopting the actual values $R_{b\ exp}$, determined on cubic samples.

– The ratios F2 (design load, $R_{b\ exp}$) / F1 (R_b form of computation theory), known values between 1.053 (semi-slab M1||) and maximum 1.067 (semi-slab M3⊥). The symbols "||", and "⊥" mean: "ribs parallel with the axis of profile I18" and "ribs perpendicular to the axis of the I18 profile", respectively.

– The drift of the experimental plastic load (F3) from the theoretical (F1) is less than 5.60% (semi-slab M1||), about 20.80% ... 17.00% (semi-slab M1⊥ and M3⊥) and 31.10% (M4|| semi-slab) and g.2 - method "B" analysis (5):

– The calculation load produces plastic articulation in the middle of steel beam profile, I18, appreciated with the theoretical values of compressive strength of concrete of the plate (F1) present deviations to the similar results obtained by adopting the values $R_{b\ exp}$ determined on sample cubes (F2). Reports F2/F1 have values between 1.003 (semi-slab D M1||) and maximum 1.013 (semi-slab M3⊥). Deviations are significantly lower than method's "A" case.

– Differences between experimental plastic load (F3) from the theoretical (F1) is less than 6.50% (semi-slab M1||), about 21.80% ... 17.90% (M⊥ semi-slab and M3⊥) and 92.70% (semi-slab M4||), which is an exception.

3.7.1 The "C" method for analysis (6):

– As in methods "A" and "B" the theoretical load producing the plastic articulation's development in the middle of the steel profile I18, appreciated the normalized values of concrete compressive strength of plate (F1) shows deviations from homologous results obtained by adopting effective resistances $R_{b\ exp}$ as determined on cube samples (F2).

– Deviations are significantly lower than in method "A". The ratios F3/F1 and F3/F2 show, also, notably grouped values. The differences between the experimental value of the plastic load (F3) and the purely theoretical value (F1) are less than 4.70% (M1 semi-slab ||), of about 19.70% ... 15.90% (M⊥ semi-slab and M3⊥) and of 89.50% (semi-slab M4||), which is confirmed in an exception.

3.7.2 Comparative values:

– Method "B" for theoretical yield capacity (F1) load assessment, the records results with 8% is lower than in method "A" and 17% is lower than in method "C". Such differences are not found to increase value. Between calculation "A" and method of calculating "C" size difference amounts to about 9% and no one is a significant result. Higher percentage are results using the methods of calculating the "A", "B" and "C" .

– The theoretical coflexures calculated using method B, using experimental forces specific to 0.80 * (ULS) - ultimate limit states and effective strength $R_{b\ exp}$ are, generally, closer to the four investigated specimens and values between 0.144 cm 0.157 cm specimen M1 and M2 sample, for values of 17 150 daN form ... 18 630 daN. The calculation method is not surprising the influence of the steel plate's position from the supporting profiles. Therefore, vertical displacement stiffness K theoretically have similar values for all four models and are in the range 118 662 daN/cm - model M2 and 119,127 daN/cm - model M3.

– The theoretical stiffnesses are higher by 917% - M|| and by 1305% - M⊥, which seriously undermines the hypothesis that the "B" method of calculation could be used in near post-elastic field.

– The M⊥ model is more flexible with 42% than M|| and not only for this reason calculation method "B" should undergo fundamental correction.

3.8. Push - test specimens:

3.8.1. The "A" calculation method shows results closest to the experiments, with deviations up to Q2/Q3 = 13.90% - and only specimen E2 ⊥ Q2/Q3 = 2.60% - sample E11||, to crack.

3.8.2. The "C" calculation method is underestimating of the bearing capacity with 22.40% - E11|| and 16.60% - E2⊥. The largest differences from the calculation are for "B" method, with values of -0.40% - E1 || -36.30%

and - E2⊥, and drastic underestimation of the sliding capacity of a connector.

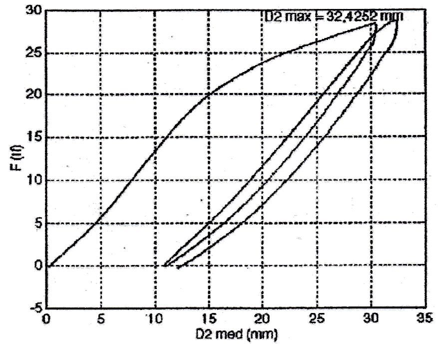

Fig. 3. P4, Exp. 3, curve F – $D_{2\,med}$

Thus, between the values of calculation methods provided by "A", "B" and "C" there are known differences A / B = 71 ... 78 %, A / C = 32 ... 36 % and, depending on the assumptions used in design, either method "A" or method "C" are recommended. Fig. 3 shows the hysteretic curve describing the evolution of the static vertical displacement in the post-elastic range, for model M4.

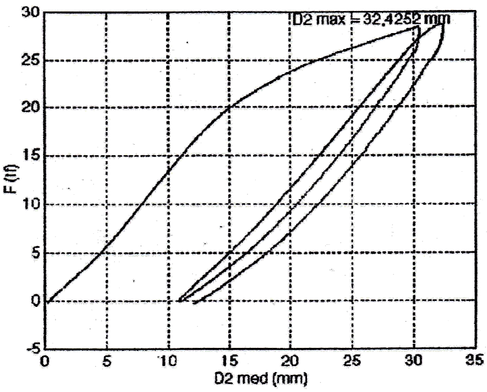

Fig. 4. Model M4, Exp. 2, dynamic

In Fig. 4, the experimental results show the same pattern. These results are intended for the improvement of current design standards.

4. CONCLUSIONS

The phenomenon of transmitting the relative plate-beam sliding to the anchorage connector is characterized especially by the structural conditions, which are including: the material's characteristics (steel, connectors, concrete), extra reinforcement connectors depending on the layout of the plate ribs, the bearing beam, the plate's ribs arrangement and the steel profile direction.

Regardless of the direction of the ribs, the capacity of a connector to resist the relative sliding is around 9000 daN. An increase of 35% can be detected for the reinforced connectors located in the ribs disposed parallel with the steel beam. The value of 9000 daN / connector provides a good assessment for calculations according to (4), with a maximum error of ± 3%.

The capacity of the plate – beam to bear the sliding force can be increased by 35% with confining connectors, only if the ribs are parallel to the supporting beams.

The parallel arrangement of the ribs with respect to the supporting beams leads to a coflexure (for a plate span / plate length = 0.50) smaller by about 50%, at the serviceability limit state, as compared to the situation when ribs are perpendicular to the laminated profiles.

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