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SHEAR CONNECTION IN STEEL AND CONCRETE COMPOSITE TRUSSES

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Abstract*. Distribution of longitudinal shear along an interface between steel and concrete of composite trusses from elastic phase up to plastic collapse is investigated. Based on experimental investigation the corresponding 3D FE numerical model has been developed using ANSYS software package. Both floor and bridge steel and concrete composite trusses are analysed. While in the former the plastic redistribution of longitudinal shear flow is of the main interest, the form of non-linear elastic distribution of the longitudinal shear is important particularly for class 3 and 4 sections, fatigue behaviour and non-ductile shear connectors, all important notably in design of bridges. Numerical results emerging from parametrical studies are compared with Eurocode 4 provisions. Extensive study also deals with influence of densification of shear connectors above truss nodes. Finally some recommendations for practical design are suggested.*

1 INTRODUCTION

Distribution of the shear flow between steel truss and concrete slab along span of a composite truss girder has been investigated since sixties both experimentally and theoretically. Eminent experimental research in USA and Canada was performed e.g. by Galambos and Tide [1], Iyengar and Zils [2] Brattland and Kennedy [3], Kennedy and Woldegiorgis [4], Viest [5] and enabled extensive use of composite trusses as large span floor beams. In nineties the research by Neal and Johnson and SCI publication [6] led to design recommendations showing wide range of design aspects important for composite steel and concrete trusses. In compliance with these recommendations ***the plastic design*** can be done identically as for a common plate girder, including the design of a steel-concrete shear connection, provided the shear connectors are ductile and bending rigidity of upper steel flange of the truss is sufficient.

In an ***elastic design of shear connection***, however, needed for class 3 and 4 cross sections, rigid shear connectors with respect to their limited deformation capacity and generally required for bridges the highly non-uniform distribution of longitudinal shear flow in a composite truss girder due to transmitting of the shear forces to the concrete slab within truss nodes needs to be taken into account. This case was indirectly dealt with by Johnson and Ivanov [7] and introduced into Eurocode 4 (EN 1994-2, or ENV 1994-2 in more detail). The Eurocode provides formulas for the local effect of a concentrated longitudinal force and distribution of the longitudinal shear force into local shear flow between steel section and concrete slab. The worked example using these formulas and a comparison with non-linear analysis employing ANSYS software was presented by Machacek et al. [8]. Extensive numerical analysis of composite trusses both in elastic and plastic region was presented by the authors in [9], embodying also experimental investigation of two floor 6 m span composite trusses. The numerical model described below proved to correspond excellently with the tests results and enabled to analyze more than 30 variants of shear connections of simple trusses having various load-slip relationships obtained from previous research [10].

In this paper the main principles of the shear flow distribution along concrete-steel interface of composite floor trusses are summarized and analysis of typical composite truss bridge girders is described in a detail. The attention is paid to both bridge trusses with and without gusset plates, influence of rigidity of upper steel flange and densification of shear connectors above truss nodes. The Eurocode 4 proposals are verified and recommendations for practical use are suggested.

2 FEM MODEL

GMNA (geometrically and materially nonlinear analysis) FEM model using ANSYS software was developed and verified with tests [9]. The special 3D reinforced concrete elements (SOLID65) were used for the concrete slab, while steel bottom chord and web bars were modelled by beam elements with appropriate cross section (BEAM24) and upper chord of the steel trusses was composed of shell elements (SHELL43). All steel finite elements enable elastic-plastic analysis (furthermore with large deflections), concrete elements involve smeared reinforcement, crushing and cracking (for shear transfer in opened and closed cracks ANSYS coefficients C1 = 0.3, C2 = 0.6 were used). Shear connection was modelled by non-linear springs located uniformly in required distance along span and placed at a suitable spot between the anticipated shear connectors and concrete slab (e.g. at the centre of gravity of holes in perforated shear connector or at base position of headed studs). Nonlinear two node spring element COMBIN39 was employed which makes possible any nonlinear relation between force and extension to model correctly shear forces in direction of girder axis. Vertical and transverse displacements of concrete slab (perpendicular to the girder axis) at springs positions were defined being the same as for the shear connector/steel flange (i.e. no uplift effects were considered in the analysis).

The model was applied to experimental trusses employing real steel and concrete properties and load-slip diagram of used perforated shear connector. Both numerical and experimental central deflections and slips between steel truss and concrete parts were nearly identical.

Following numerical studies were performed with characteristic values of material properties. Simplified bilinear stress-strain diagrams of steel and multi-linear for concrete were used and are shown, e.g. for steel S355 and concrete C25/30, in Fig. 1.

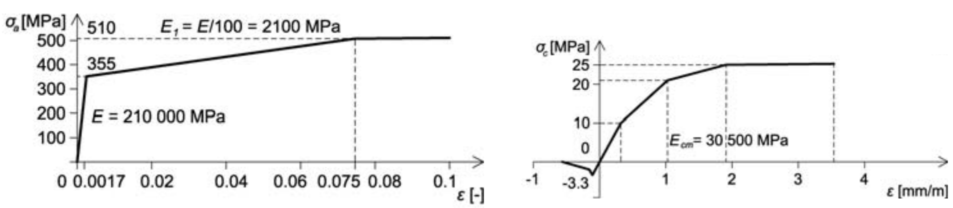


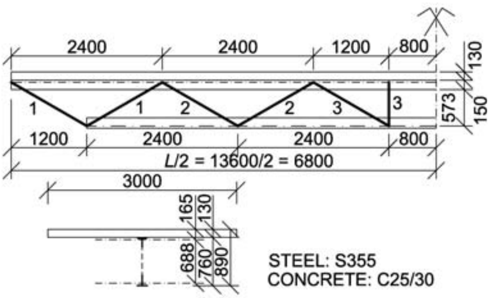
Figure 1: Example of stress-strain diagrams of steel and concrete used in parametric studies.

3 NUMERICAL STUDIES

First the principal results of large parametrical study concerning a typical floor composite truss are summarized, followed by an investigation of important parameters of typical bridge composite trusses. Generally the uniform line loading was imposed on the concrete slab within steel chord width along entire span and gradually increased up to a truss collapse (corresponding to the maximum loading shown in following figures). Standard Newton-Raphson iteration was employed while crushing function for concrete was finally not applied due to problems with proper convergence of results in final loading steps (believed, however, to have no substantial influence on results).

3.1 Floor composite trusses – summary of results

Parametrical study covering more than 30 various types of shear connection differing in load-slip diagrams and distribution of shearing capacity (i.e. densification of shear connectors above truss nodes) was published in [9] for the typical floor truss shown in Fig. 2 (upper chord of the steel girder is designed as ½ IPE 300, bottom chord ½ IPE 330, diagonals # 1 as 2L 80x8, # 2 as 2L 60x6 and # 3 as 2L 50x5).



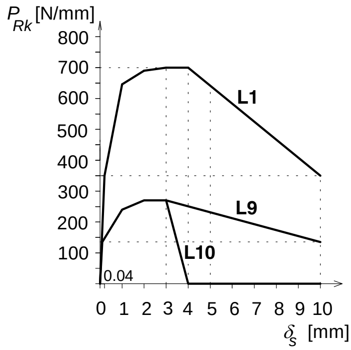
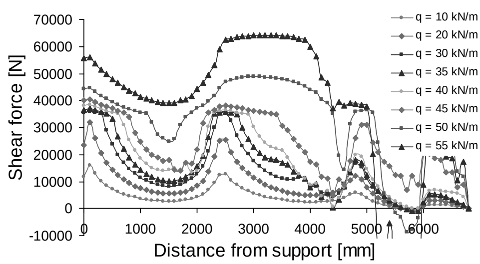


Figure 2: Analyzed floor truss (half span shown) and load-slip diagrams of connections L1, L9 and L10.

In this paper only the connection L1 corresponding to ***full shear connection*** (the full truss flexural strengths correspond to v_{sf} = 304 N/mm or 436 N/mm taking yield stress f_y or ultimate stress f_u into account respectively) and L9 corresponding to ***partial shear connection*** are shown for illustration (L10 with limited ductility), where the diagrams demonstrate typical behaviour of perforated shear connectors [10] with ascending part substituted by trilinear representation.

In following figures the shear forces in shear connectors are shown ***for one half of the truss girder***. The curves show values for load steps increasing up to the numerical collapse value (given by maximal value in captions), for which the support slip in the shear connection is provided. Either the shear force in shear connectors (placed by 100 mm) or shear flow for comparison with Standard approach is given at the vertical axis while the distance from support at the horizontal one (truss nodes are in distances 2400, 4800 and 6000 mm).



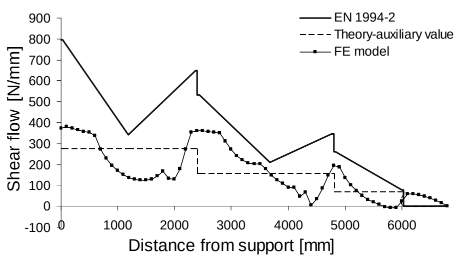


Figure 3: Shear forces in L1 connectors (per 100 mm): FE model (end slip δ_s = 0.4 mm) left, comparison with Eurocode 4 for loading 38.2 kN/m (elastic resistance for fabrication with propping) right.

In all connections the plastic redistribution of shear forces occurs soon after reaching the second (less stiff) segment of trilinear ascending part of connector P-δ_s diagrams. In L1 connection (Fig. 3) comparison with Standard approach is shown which gives apparently conservative values when commencing plastic redistribution takes place. At early elastic behaviour, however, the Standard solution gives good approximation of peak values (see [9]). In L9 case (see Fig. 4 left) the connection decides about resistance of the composite truss girder (see the decreased collapse value from 55 to 46 kN/m).

Insufficient ductility of the shear connection (case L10) results into even more decreased resistance (41 kN/m) and the most stressed shear connectors near support are getting, near the collapse loading, into descending part of the *P*-δ_s diagram, see Fig. 4 right.

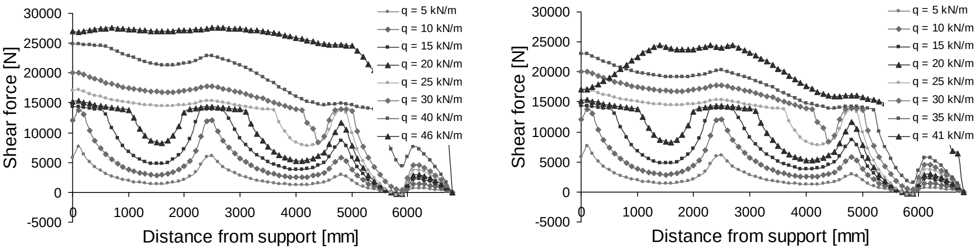


Figure 4: Shear forces in connectors (per 100 mm): L9 (δ_s = 3.2 mm) left, L10 (δ_s = 3.6 mm) right.

Densification of shear connectors above truss nodes (Fig. 5) in the peak shear flow areas was also investigated for the above floor trusses, resulting in recommendation for optimal arrangement. The parametric study showed for triple connector density above nodes the optimal extent of $d/D \approx 0.25$ (i.e. a quarter of node distance). More details are available in [9]. Care should be taken for adequate increase of the total shear flow in the densified areas (15 % in Fig. 5), which strongly depends on the increased shear stiffness (while accompanied decrease of the shear flow in non-densified areas is negligible in this case).

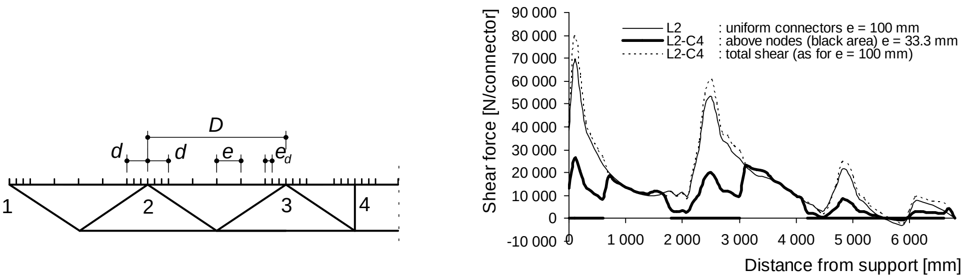


Figure 5: Densification of shear connectors for loading 38.2 kN/m (elastic truss resistance).

3.2 Bridge composite trusses

Realized composite bridges were chosen for detailed parametrical studies of some important parameters. First a special composite truss bridge without gusset plates common in Central Europe as a motorway overbridge and second a classical railway composite truss were investigated.

3.2.1 Bridge truss without gusset plates

Only modification of a real application (which is fixed at the stiff support blocks and has more diagonals) is shown in Fig. 6. The span of the truss is 21 m and steel members are from flats only [mm]: upper flange 250x20, diagonals 250x40, bottom flange 300x40. Headed studs with 19 mm diameter located in 3 parallel rows and longitudinally in distance of 200 mm were considered for shear connection, giving characteristic shear strength 3x77100/200 = 1156 N/mm (T1). As an alternative the shear strength was decreased to 70% (T2), giving 809 N/mm, Fig. 7. Nevertheless, both alternatives represent **full shear connection** (corresponding to shear flow 575 N/mm, taking f_y into account) as required for bridges.

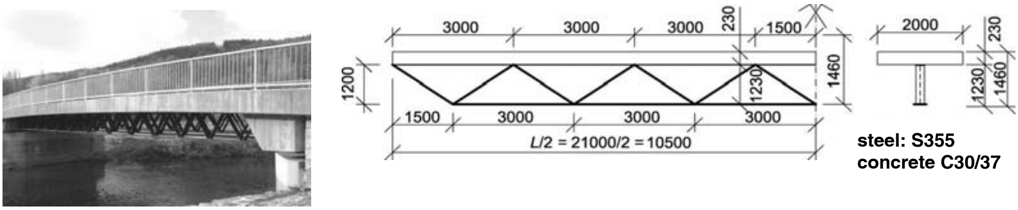


Figure 6: View of the bridge and modified setup for analysis.

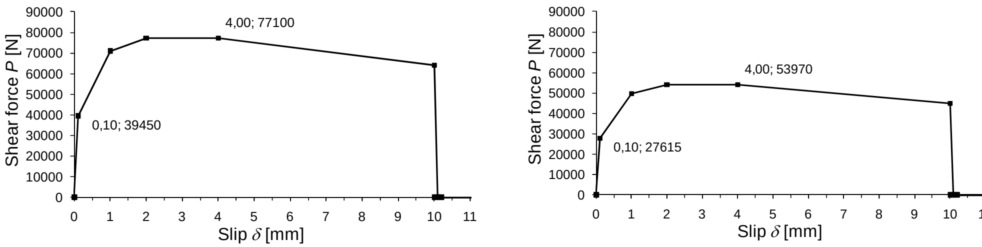


Figure 7: Load-slip diagrams of shear connections T1 (left) and T2 (right).

As in previous chapter the shear forces in shear connectors are shown for one half of the truss girder and the curves show values for load steps increasing up to the numerical collapse value (given by maximal value in captions), Fig. 8. At the vertical axis the shear force in shear connectors is shown while at the horizontal axis the distance from support is given.

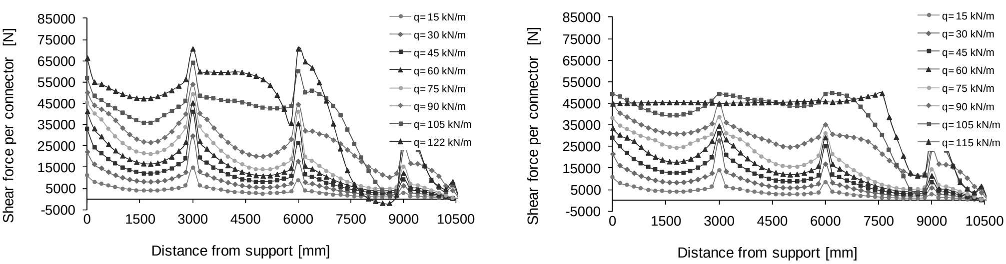


Figure 8: Shear forces in connectors (per 200 mm): T1 (left), T2 (right).

The less stiff shear connection T2 demonstrates less distinctive shear peaks but some decrease of collapse load in comparison to T1 connection (note that Eurocode 4 plastic resistance corresponds to 112,5 kN/m, elastic one to 102,0 kN/m, for fabrication with propping), which is due to final plastic redistribution of shear forces in extremely loaded shear connectors. Shear flow according to numerical FEM and simplified Standard procedure (EN 1994-2) for loading 75 kN/m (near the design loading) is shown in Fig. 9. Within the Eurocode approach the effective width available (b_{eff} = 2 m) and twice the half flange depth $e_d = 2e_v$ = 20 mm (because of no gusset plates) in accordance with the Standard were applied.

It follows that Eurocode approach is conservative also in this case and the more conservative the less rigid the shear connectors are. Agreement of FEM and Eurocode approach is always better at lower loading (considering high initial stiffness of the shear connection).

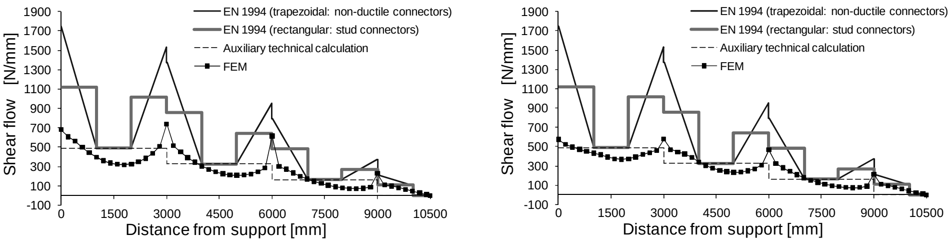


Figure 9: Comparison with Eurocode 4 approach, loading 75 kN/m: T1 (left), T2 (right).

Another investigation pointed to importance of upper steel chord stiffness in this type of truss. The thinner chord flange the higher node shear peaks must be expected. A great deformation of the chord eliminates the transfer of the shear force, which is then transmitted by the connectors directly at the nodes only. The shear force peaks above nodes depending on upper chord thickness t [mm] of this truss type is shown in Fig. 10 (note, that the thickness in Figs. 8 and 9 is t = 20 mm).

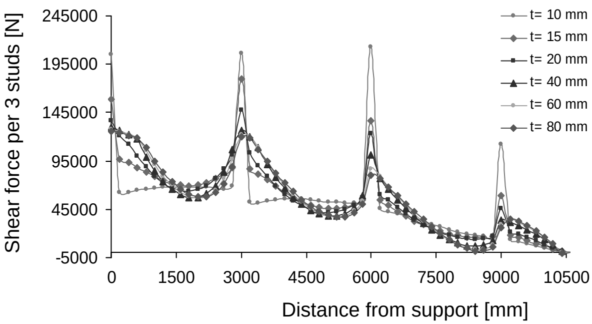


Figure 10: Shear forces in shear connection T1 (distance of connectors 200 mm), loading 75 kN/m.

3.2.2 Large span bridge composite truss

Geometrical parameters of a realized railway bridge truss with span of 63 m were used in the following studies, Fig. 11. Headed studs with 19 mm diameter located in 4 parallel rows and longitudinally in distance of 400 mm were considered in the study as a basic arrangement, giving characteristic shear strength 4x91400/400 = 914 N/mm, while full shear connection ranges from 809 N/mm to 1023 N/mm (considering non-uniform chord cross section and taking f_y into account). The FE modelling as above and load-slip diagram by Oehlers and Coughlan [11] given in Fig. 11 were used.

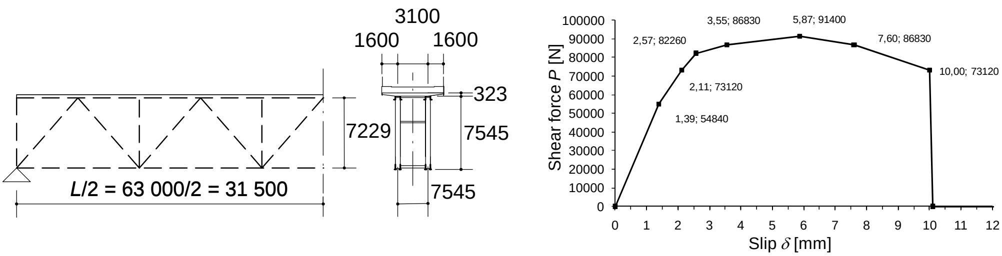


Figure 11: Geometry of the composite truss bridge and load-slip diagram of studs.

The simplified Eurocode 4 calculation gives elastic characteristic resistance of the truss 269.5 kN/m and plastic one 283.3 kN/m. The FEM distribution of shear forces per connector in basic uniform arrangement and deflection of the truss at midspan are shown in Fig. 12. After commencing plasticity in the bottom chord of the truss (at approx. 270 kN/m) and following plastification of shear connectors (after attainment of shear force at approx. 82 kN) a rapid plastic shear flow redistribution yields into truss collapse at 325 kN/m. Upper steel chord had to be strengthen between the first two nodes to have sufficient rigidity (see chapter 3.2.1) ensuring supposed transfer of the shear forces.

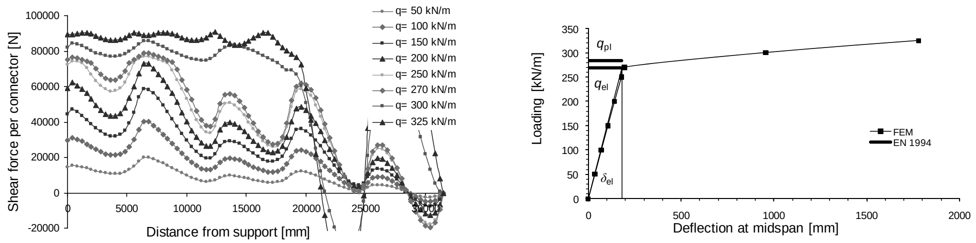


Figure 12: Shear forces per connector (basic uniform arrangement) and midspan deflection of the truss.

Comparison of shear flow distribution with Eurocode 4 approach at early elastic loading (100 kN/m) and near to design loading (200 kN/m) is shown in Fig. 13. The Eurocode distribution in the peaks is shown for both non-ductile shear connectors (inclined, trapezoidal distribution) and ductile ones (rectangular distribution). The local shear in accordance with the Eurocode is distributed on the length $L_v = e_d + b_{eff}$ = (1500 + 2 x 148.5) + 3150 = 4947 mm and gives very conservative values. The decisive role plays the effective width b_{eff}, which is limited by the concrete slab width available (in contradiction to assumption in [7]).

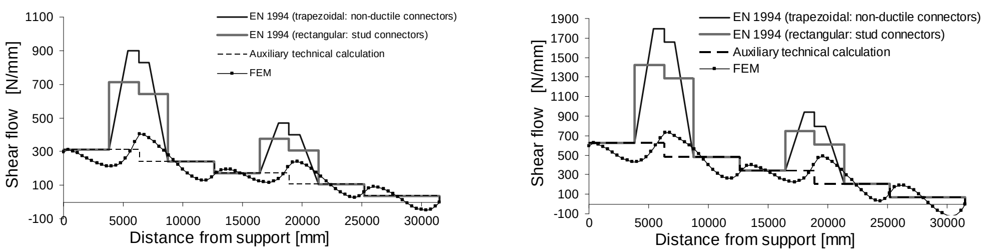
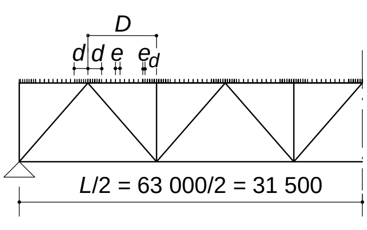


Figure 13: Comparison of FEM shear flow with Eurocode 4 approach at loading 100 kN/m (left) and 200 kN/m (right).

Densification of the shear connectors above truss nodes covering shear peaks was also investigated. Quadruple densification was assumed in three various lengths *d* and denoted as D1, D2, D3 (Fig. 14).



Basic arrangement:
4 parallel studs, e = 400 mm
Densification:
D1: 4 parallel studs, e_d = 100 mm, $d = D/4$
D2: 4 parallel studs, e_d = 100 mm, $d = D/5$
D3: 4 parallel studs, e_d = 100 mm, $d = D/6$

Figure 14: Stud densification geometry above truss nodes.

With narrower densified area the shear forces per connector are, of course, higher as seen from the distribution (Fig. 15) and also increased total shear above nodes is obvious. Extent $d/D \approx 0.25$ (i.e. a quarter of node distance) seems to be optimal even in this truss. The increase of the total shear flow in the densified areas (dotted line) reached in D1 case 97 % (in D3 even 129 %), accompanied by a corresponding decrease of shear flow (approximately to one half) in non-densified regions.

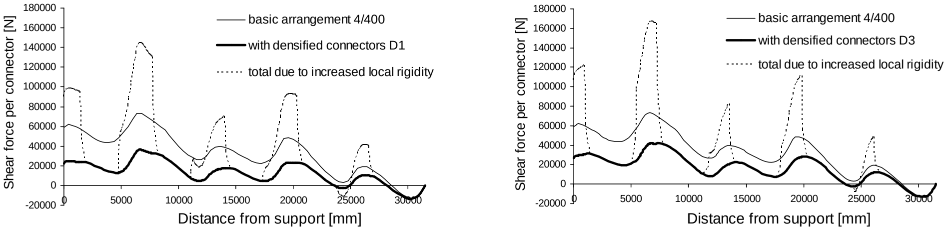


Figure 15: Densification D1 ($d = D/4$) left and D3 ($d = D/6$) right at loading 200 kN/m.

4 CONCLUSION

Eurocode 4 approach for elastic local effect of concentrated longitudinal shear gives reasonable solution even in case of truss node forces. However, the approach depends substantially on effective slab width and the real available ones results into overly conservative results. Likewise shear connector and steel flange rigidities influence considerably the magnitude of shear flow peaks above truss nodes in elastic region, which are not covered by the Standard approach. Densification of shear connectors above nodes implicates considerable redistribution of the shear flow (attracting the flow to rigid areas).

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