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mesh, which is suitably curved and welded on the top of the steel plate in some places. It also acts as an standard reinforcements also. First phase of the paper shows the pull out tests preparation.

Namdeo Adkuji Hedaoo, Laxmikant M Gupta and Girish N Ronghe [4] created the slab created by composite interaction between the deck with embossments to improve its shear bond characteristics. The results are compared and graphically represented the load-deflection curves, load-end slip curves and failure modes subject to imposed loads. The shear span used here is 300, 375, 450, 525, 600 and 675. The end slip at the ultimate load failure is between 2 to 3.6 mm. As the shear span length increased, the longitudinal shear stress of slab decreased. The results are shown below:

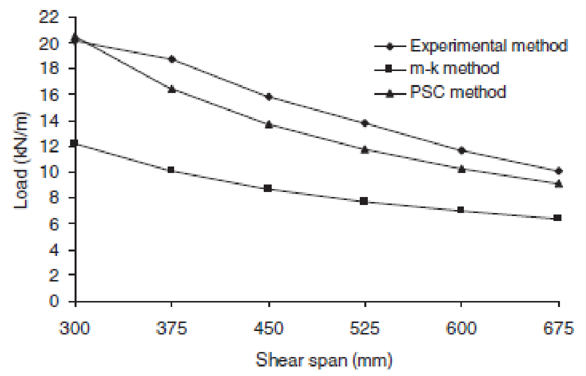

Figure 3: Failure/design load to shear span under flexural loading.

[5] Investigated the behaviour composite deck slab under hogging moment. The steel deck contributes to the negative moment capacity to the positive moment capacity. The simple analytical model predicts the ultimate negative moment capacity and moment–curvature relation of the composite slabs. A certain degree of ductility was observed in the slab which depends on amount and type of reinforcement. The ductility can also be improved by using steel reinforcement of higher ductile, high-yield mild steel. The ductility observed here is based on a constant moment region which is less severe than that of a peaked moment region commonly experienced.

P. Patil, M. Shaikh [6] presents 3D numerical models [illegible] SCC beams to simulate their behavior, with emphasis [illegible] the beam slab interface. Simulations were carried [illegible] using version 14.0 ANSYS.
The objective is to verify the influence of [illegible] amou[illegible] diameter and height of shear connectors in composi[illegible] beams. These verifications were made by analysis [illegible] longitudinal slip in the slab-beam interface, the vertical displacement at mid-span and the bearing capacity of composite beams.

Table 1: Result with variations of height of studs.

Parameter	Φ(mm)	F_{max}(kN)	U_{max}(cm)	d_{max}
H=76mm		506.9	9.24	0.0188
H=88mm	19	481.3	6.48	0.0143
H=102mm		481.4	6.54	0.0149

Table 2: Results considering the change in diameter

Parameter	Φ(mm)	F_{max}(kN)	U_{max}(cm)	d_{max}
Φ =16mm	102	437.68	3.84	0.0133
Φ =19mm	102	481.46	6.54	0.0149
Φ =22mm	102	506.28	9.29	0.0151

[7] Evaluated the behavior of Y-type perfobond rib shear connector(A new type of connector) which is superior in shear resistance and ductility than conventional perfobond rib shear connector. Various types of the proposed Y-type perfobond rib shear connectors are examined to evaluate the effect of design variables such as strength of concrete, transverse rebar, thickness of rib having Y-shape angle. It was also proven that Y-shape angle are effective on shear resistance and ductility. It is indicated that Y-type perfobond rib shear connector has higher shear resistance and ductility than the conventional perfobond rib shear connector.

The same relative slip, the Y-type perfobond showed 12% higher shear resistance. The ductility of the Y-type perfobond rib shear connector was 255% of that of the conventional perfobond specimen and shear resistance has been increased by 8%. S. Ranković, D. Drenić [8] researched the mechanism of failure and basic criteria for defining of the shear connectors strength on composite slabs with profiled sheet.

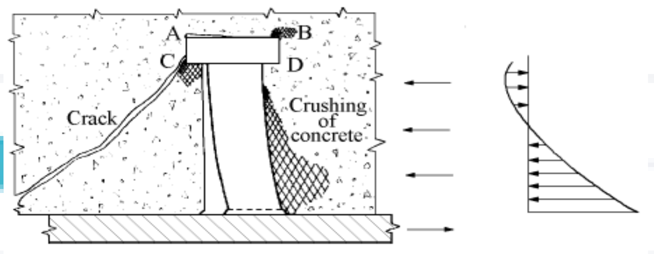

Figure 4: Failure model of headed stud

The shearing force at composite beams with profiled sheet is not introduced directly through the base of the shear connect, but is transferred onto the shank of the shear connector. The increased load produced crushing of concrete in front of the shear connector and transfer of shear force exclusively via bending. The maximum [illegible] is re[illegible]r due [illegible]ure of concrete or due to the fai[illegible]e of the shear [illegible]nnecto[illegible]ove the weld.

[illegible] Marimuth[illegible] S. Seetharam[illegible]a, S. [illegible]rul Jayachandrana, A. [illegible]hellapp[illegible], T. Bandy[illegible]adhy[illegible], D. Duttab [9] expe[illegible]ented [illegible]e study of th[illegible]hear b[illegible]d behavior of the embo[illegible]d co[illegible]osite deck sl[illegible] under [illegible]ulated imposed [illegible] using [illegible] The [illegible]xperime[illegible] evaluation of m–k values for composite deck slabs were carried out using cold formed profiled sheets with rectangular dishing type embossments. The shape, size and frequency of the embossment, shear spans of 320 mm, 350 mm and 380 mm, 850 mm, 950 mm and 1150 mm are adopted. The load deflection behavior and slip behavior of the slabs were experimented and the results were compared accordingly. The behavior of the sheet nearer to shear span depends on the shear bond failure while the larger shear span depends on the flexure capacity of the slab.

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Table 3: Details of shorter shear span loading and its behavior

No	Shear span L_s(mm)	Failure load (kN)	Behaviour
1	320	55.625	First stage: Shear cracks were formed near the loading point and sudden drop in the capacity.
2	350	52.191	Second stage: Carried additional load by reinforcement mesh provided at centre of the concrete. Flexure also at loading point.
3	380	47.340	Slip: Slip was observed from the early stage of loading and the rate of slip was higher after first stage.

Table 4: Details of longer shear span loading and its behavior

No	Shear span L_s(mm)	Failure load (kN)	Behaviour
1	850	22.612	Flexure cracks were formed in between the loading points accomplished by a sudden drop in the capacity.
2	950	26.920	Additional load was resisted by nominal reinforcement mesh provided at centre of the concrete.
3	1150	16.391	Slip: Rate of slip is comparatively higher.

A. Gholamhoseini, R.I. Gilbert, M.A. Bradford, Z.T. Chang [10] proposed longitudinal shear failure between the concrete and the steel decking the most common type of failure at the ultimate load. The bond–slip relationship of each slab was determined during the testing. The values of maximum longitudinal shear stress is also calculated. Post-cracking bending moment capacity in all slabs was achieved, but the maximum flexural capacity was controlled by slip at the concrete–steel interface in the shear span and the plastic flexural capacity of the slabs is neglected. The numerical model is also shown for accuracy and reliably prediction of the results.

3. Discussion

Based on the review of the above researches it is observed that the slippage in the composite deck slab can be controlled by introduction different types of shear connectors between the steel and concrete composite. However, with the increase in loading and bending moment in the composite deck slab, it was observed that the support connection is highly critical when it comes to reducing the slippage in the slab. The tests are carried out from the literature survey results mainly in the load deflection relationship or load slippage relationship. Because of the negligible friction between steel and concrete, and also the shear behavior between this is very complex the definite connection is therefore not possible and the values are taken from the past or it is experimented.

4. Conclusion

Form the literature review it is concluded that, the shear transfer prediction and calculation is very complex and iterative. There is no bonding between these two materials to act as one. Hence it is required to provide a connection with sufficient anchorage so as to induce a proper load transfer pattern in the composite deck slab along with the shear connectors.

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References

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