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Howard D. Wright

Milan Veljkovic

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TOWARDS A NUMERICAL PROCEDURE FOR COMPOSITE SLAB ASSESSMENT

Howard D. Wright[1] and Milan Veljkovic[2]

Summary

Composite slab design relies upon adequate shear bond resistance between the steel and concrete. This is achieved by friction between the materials and the mechanical interlock of embossments pressed in the steel. The behaviour of this shear bond resistance is complex and is obtainable only from model tests. This paper explores the potential for numerical modelling to provide the required design information.

Introduction

Composite floor decks are a popular method of providing convenient and light slabs in steel framed structures (1). The light gauge profiled steel sheeting provides both formwork for the wet concrete and reinforcement to the final slab. The major design constraint for the service condition is the shear bond resistance between the concrete and steel. Shear bond resistance comprises a combination of friction occurring between the two materials and bearing of the concrete on a pattern of embossments pressed in the steel sheeting.

Despite considerable research into the nature of the shear bond resistance of composite floor decks since the 1970s the latest Code method of deck design (2) still relies upon performance testing of full scale specimens. Various design methods have been proposed (3,4,5,6) but all rely upon tests to provide data from which behaviour may be extrapolated. The tests may be full scale or small scale model specimens.

There has been attempts to model the behaviour using numerical analysis (7,8). This is an extremely complex problem, especially if the individual embossment behaviour is to be modelled. To date, the models produced are not at a stage whereby general design, using them, may be considered.

The aim of the study has been to use experimental data produced in Strathclyde in a numerical model developed in Lulea. This provides greater confidence in the numerical model and new insight into the behaviour of composite slabs failing in a ductile longitudinal shear mode.

1. Professor, Department of Civil Engineering, University of Strathclyde, Glasgow, Scotland
2. Postgraduate student, Department of Steel Structures, University of Lulea, Lulea, Sweden

Previous work on shear bond in composite slabs

Schuster (9) carried out early work on the bond developed between steel sheeting and concrete. Porter and Ekberg (10) furthered this work and laid the foundations for a design method that has now been adopted widely. The design method is described more fully in reference 1 and relies upon full scale bending tests. Two coefficients m_r and k_r are obtained from tests on upwards of 8 specimens. These are then used in a semi-empirical expression which allows the interpolation of load span limits for slabs of the same geometry but varying concrete depth. The use of full scale tests to determine design parameters is defined, here, as a macro level investigation. See Fig. 1.

The m_r k_r method relies upon the testing, at full scale, of several slabs. Many researchers have investigated the use of model tests to reduce costs. Schuster's original work (9) involved the use of model specimens which have now become known, generically, as Push-off tests. Abdel-Sayed et. al., (11), Stark (12), Jolly and Zubair (13), Daniels (14), Patrick (6), and Veljkovic (15) have all described differing forms of push-off or small scale bending test. Li and Cederwall (7) describe a small scale bending test which has some similarities to the push-off philosophy.

These tests have developed in complexity since the early 1970's but have mainly relied upon the application of a direct shear load between the concrete and steel on small samples of real steel sheeting and concrete. To simulate the reaction load of a slab support many of the test models include the application of normal forces. Despite the increasing complexity of these tests they are, in the authors' opinion, unable to mirror the complex interactive behaviour of slab bending and shear that occurs in the real slab.

Push-off tests are, however, able to provide qualitative information on performance especially when comparisons between various types of deck are made. This was the theme behind the work of Jolly and Zubair (13) in their study of a variety of embossment shapes. Daniels (14) also made comparative conclusions between types of deck and defined two types of generic behaviour; ductile and brittle.

Veljkovic (15) is further extending the development of push-off tests by using a relatively complex test specimen where the steel sheeting is put under a tensile strain whilst the concrete is pushed over it. This reduces the resistance of the embossment interlock and may, more closely represent the behaviour occurring in a real slab. Parameters obtained from the push-tests have been used with some success in a numerical model of the slab. The numerical analysis is described more fully later in this paper.

Push-off tests may be described as a meso level investigation. More detailed study of the individual embossment behaviour may be described as a micro level investigation. Essawy (16) has used a very basic push-off test model to investigate detailed embossment behaviour and validate numerical studies of the behaviour of single and groups of embossments. These push-off tests are described more fully later in the paper and results from the tests are used with the numerical model developed by Veljkovic (15).

Full scale tests

A major development in modern deck design has been the use of a re-entrant portion in the profile geometry. This creates a vertical key holding the embossments and concrete together. In addition the re-entrant portion of geometry was thought to provide supplementary frictional resistance. Early Holorib (17) decks relied exclusively on this mechanism for their bond strength.

In order to establish the performance of a profile using only a re-entrant portion to provide shear bond Ward Building Components (18) were asked to roll some special samples of their popular Multideck 60. These samples were identical to the normal embossed profile shown in Fig. 2 but were unembossed.

Two slabs were cast with their geometry being described in Fig. 3. Slab 1 used the unembossed profile and slab 2 the embossed version. Average material properties are also presented in this figure. In both cases mesh reinforcement was fixed in one half of the slab. This was to strengthen that half of the slab and ensure that failure occurred in the other half. This enabled 28 electric resistance gauges to be fixed in the weaker section of slab in the knowledge that the failures would be fully recorded.

Slab 1 was tested with three cycles of loading using a two point load system. The first load cycle to 30 kN (45% of the ultimate load) was stopped when cracking noises began to emanate from the specimen. This was thought to signal the breakdown of bond. The second load cycle to 47 kN (70% of the ultimate load) was stopped when diagonal shear cracks became visible under the load points. In the final load cycle deflection control loading continued until the specimen was fully failed (67 kN).

A similar load history was carried out for Slab 2 and revealed loads of 40 kN (46% of ultimate load) for concrete debonding, 71 kN (81% of ultimate load) for the onset of diagonal shear cracks and an ultimate load of 87 kN.

It was interesting to note that the slab formed with the unembossed sheeting carried 77% of the ultimate load of the slab with embossed sheeting. Other specific observations are as follows:-

- Final failure of the slab with unembossed sheeting was caused by buckling in the webs, whereas upper flange buckling occurred in the slab using embossed steel sheeting. The embossments appear to have strengthened the out-of-plane stiffness of the web plates.
- At failure the concrete in the first slab had almost completely separated from the steel with the nib of concrete in the re-entrant portions breaking over a significant proportion of the shear span. This may indicate that the re-entrant portion and the friction generated at the support has less effect on the ultimate capacity of the slab than has been previously supposed (6).
- In the second slab the two materials remained largely connected by re-entrant locking. The stiffening of the webs and the additional resistance to separation provided by the embossments were thought to be largely responsible for this. This is particularly the case given the complimentary deformation of the two flanges towards to concrete which is caused by the deformation of the web away from the concrete as it lifts over the embossments.

- Damage to the lower portion of the concrete at the base of the embossment signalled that this is where most mechanical bond was being generated.
- Strain gauge readings in the web showed a very different pattern between the two slabs. In the slab reinforced with plain steel a linear variation of strain occurred between the highly stressed lower flange and the less stressed upper flange. However in the embossed web very little strain was recorded. This is shown in figure 4. It would appear that the embossments create significantly reduced in-plane stiffness and act almost to puncture the web plate with holes. The web could be likened to the bellows of an accordion.

Push-off tests

The results of the slab tests provided information that suggested that the re-entrant portion of the profile was significant in improving shear bond behaviour. In addition it appeared from the tests that the bottom part of the embossments were subjected to most load. To further investigate this effect a series of push tests were carried out.

These were designed to be as simple as possible whilst still being able to provide comparative data between various types of profiled steel sheeting geometry. The test layout is shown in Fig 7. The normal load of 5 kN was applied through the torquing down of bolts and measured with a load cell. It represents a typical reaction force present at the support position in a slab.

Two major parameters were varied in the tests carried out; the presence or absence of embossments and the effect of the re-entrant portions. Ward 60 profiles were used in the first of these comparisons. In the second a variety of profiles where used. To eradicate the effect of the re-entrant portions the undercut sections of the profiles were filled with modelling clay. A measure of the extent of the re-entrant portion was taken as the cross section area of the concrete contained under the re-entrant portions in each pitch of the profile.

In the tests the failure occurred by the concrete overriding the embossments. Scoring of the steel and slight crushing of the concrete was observed at the base of the embossments. This was similar damage to that observed in the full scale slab tests. Little damage occurred to the re-entrant portions in the test on the unembossed Ward 60 profile but in all other tests the re-entrant nib of concrete was found to be broken following failure. The load recorded in the cell remained constant until the final stages of the test when the block had lifted and moved several millimetres.

These tests show the importance of simple friction. The unembossed Ward profile sustained approximately 50% of the load of the embossed profile. It is also shown that the re-entrant portion has a significant effect, accounting for improvements of between 63 and 88% on identical profiles with the re-entrant portion filled with clay. The enhancement associated with the re-entrant portion appeared to increase with the area of concrete contained under it. An approximate linear relationship was evident from this limited number of tests.

It may also be surmised from the damage incurred by the concrete and embossment steel that most shear resistance would appear to be derived at the base of the embossment. Given the fact that the normal load did not vary until the vary late stages of each test it may also be surmised that the overriding of the concrete was made possible by the local deformation of the web plate.

Numerical study

The bending test of the composite slab containing the sheeting with the embossments has been numerically modelled in 2D. The calculations have been performed with the DIANA finite element package version 5.1.

One of the main purposes for the numerical study was to better model the failure mechanism of the full scale tests. There are four parameters that influence resistance of the slab:-

- friction at the support
- mechanical interlocking in the shear span
- reduction of the mechanical interlocking due to large strains in the sheeting
- local buckling of the sheeting.

The push-off tests have been used to provide quantitative values for the friction at support and the mechanical interlocking in the shear span. Rather more qualitative assumptions have to be made for two other parameters which are not experimentally examined. It has been shown by Veljkovic (15), that a reduction in the mechanical interlocking occurs when the steel is subject to large strain. The deformation of the cross section of the embossment during the overriding of the concrete deck is qualitatively shown in Fig. 6. The effect of tension strain across the embossment is clearly identified and is compounded by the "accordion effect" observed in the full scale tests. Furthermore, the deformation pattern of the embossment illustrate the complexity of the stress state in the web.

In modelling the mechanical interlock the strain level in the sheeting and the amount of the slip should be incorporated. The experimental study required to establish this micro level behaviour quantitatively is an ambitious exercise. It would also be difficult to implement in FE code. Therefore, it is assumed here that the reduction function depends only upon strains. In the finite element implementation it is possible to distinguish part of the slab that slips over the sheeting and only for that part the reduction for mechanical interlocking is introduced. The actual reduction has been based on experience with other type of a sheeting profile (15).

Local buckling is also a very complex modelling problem and also requires further study at a micro level. The biaxial state of stresses in the top flange, the influence of the concrete to the local buckling mode, the resistance against the vertical separation and the influence of embossments on the stiffness of the web are some problems that must be answered before being able to accurately model this phenomenon. However, the local buckling of the top flange may be taken into account by a reduction in stiffness of the compression flange using a pseudo effective width approach.

The finite element model

The mesh has been generated assuming three parts of the slab as shown in Fig 7. The figure is taken from the model whose load-displacement curve agrees the best with the experimental results and shows the deformed shape, exaggerated two times with respect to the model size. The displacement in the middle of the slab is 18.7 mm, maximum displacement is 20.8 mm, and the corresponding total load is 90.2 kN

One half of the slab was reinforced with the additional reinforcement which influence unsymmetric behaviour of the slab. The middle part of the slab was subject to pure bending and vertical cracking of the concrete deck is assumed to occur. The third part was assumed to be subject to high longitudinal shear and shear slip. For the sake of the simplicity in the model it is assumed that a major crack opening is positioned vertically under the applied load. This is always the case when crack inducers are used.

A discrete approach was used to model cracking of the concrete deck using node interface elements. In this approach it is assumed that the concrete between predefined cracks remain in an elastic state, therefore it has been modelled using plane stress elements with the elastic module and Poison ration of the concrete. The inelastic material properties of the concrete are allocated to the simplest interface element, the so called spring element. A brittle tension softening function has been used to model concrete after tension strength is reached, while an elastic perfectly plastic model has been used to model material properties of the concrete in compression.

Beam elements with compact I cross section are used to model the performances of the sheeting, but the rotation capacity has been varied in the critical cross-section, A in Fig. 7. The compression flange of the two adjacent elements to this position have been modelled with the non-linear elastic material, stress-strain relationship which correspond to the steel used. The local buckling has been taken into account by reducing the yielding plateau in compression and introducing a sudden drop of the resistance in the two elements. A plastic model with Von Mises yield criterion and hardening describes the material behaviour of the sheeting. The geometrical characteristics of the I profile are the same as for the sheeting, identical area and very close moment of inertia. No reduction of the axial stiffness due to accordion effect of the web has been used as this was assumed to be incorporated in the reduction of shear bond resistance described in the next paragraph.

The yielding of the steel in tension has been assumed at axial strains between 0.0016 to 0.0138 and the effect of the local buckling in a reduction of the axial stiffness of the flange to a strain level of 0.008 (providing the closest agreement with the experimental curve).

The interface between sheeting and concrete has been modelled using node interface elements. A non-linear elastic material model has been used to describe the mechanical interlocking properties. A slip-horizontal force relationship obtained, at Strathclyde University, from the push-off tests has been used as input data for non-linear elastic interface characteristics. It was assumed that three parameters dominantly influence the interaction; namely friction at support, mechanical interlocking in the shear span and reduction of the mechanical interlocking due to large strains in the sheeting. Strain at the integration points in the bottom flange of the beam elements are monitored, and the mechanical interlocking properties of adjacent interface elements are corrected prior to the next step depending upon the strain level. For the sake of numerical simplicity, the shape of the mechanical interlocking resistance curve has been kept constant and shrunk by a factor depending upon the strain in the sheeting, Fig. 8 and 9.

A small parametric study on the shape of the reduction function as a variable has been performed, as indicated by dotted lines in Fig 9. The reduction function used (number 4 in Fig 9) is chosen because it gives the closest agreement for the first 15mm of the midspan displacement, as well as the ultimate load. The Coulomb friction criterion has been used for the interface element at the support. The friction co-efficient was taken as 0.6.

It should be emphasised that the FE model used the following assumptions. The axial stiffness of the slab neglects the accordion effect in the web and the influence of cold forming. Furthermore, it was assumed that major crack is vertical instead of slightly inclined crack as observed in the experiment. These assumptions are not very influential for the short span slabs considered but in the case of longer spans this might lead to too optimistic a prediction.

Comparison between experimental results and FE calculations

The measurements obtained from experiments such as midspan displacement, end slip and strains in respect to the load have been compared with the results of FE calculations. The self weight of the slab and test set-up are included added to the measured load and the midspan displacement is extrapolated using the initial stiffness of the slab. Comparison of the load-midspan displacement throughout the load history is the most usual criteria to judge the success of the FE prediction. The accuracy of predicting the ultimate load is of course important parameter but certainly not sufficient to demonstrate the full potential of the numerical calculations.

The results of the FE calculation are based on two experimentally established parameters at the inter-face between the sheeting and the concrete deck; friction at support and mechanical interlock, is shown in Fig. 9. The agreement between experiment and numerical model is excellent for low loads and as the system goes plastic. The similar shape of the load-midspan displacement curves confirm that the cracking of concrete is very well modelled. An exact value for the ultimate load has not been established as the ductility of the system is very high.

The area where agreement is poorest is in the middle load range, A in Fig. 9. The discrepancy is most probably due to chemical adhesion which acts in the full scale test but is not measured in the Push-off test and consequently it is not included in the numerical simulation. Indeed, as shown in Fig. 10, no experimental end slip is recorded until a load of 60 kN is reached.

Conclusions

This paper has described full scale slab tests, small scale push-off tests and numerical studies investigating the shear bond resistance provided by embossments in profiled steel sheeting used in light gauge composite construction. Several overall observations may be drawn from these studies.

In-plane Embossment effect

In the experiments on the slabs it was noted that there was a significant variation in strain distribution between the embossed and unembossed webs in the profiles. For an unembossed plate the shear stiffness is high and, up to yield, linear variations in strain between the upper and lower flanges may be expected. In the embossed profile the shear stiffness is affected by the embossments. The embossments break up the web acting, in some ways, to perforate the plate. The deeper the embossment the closer the effect approaches that of a full penetration.

Out-of-plane flexibility of the web plate

Whilst the in-plane stiffness of an embossed web will be low the out-of plane stiffness is generally greater as the embossments act to provide ribbed reinforcement. This is especially true of embossment geometries that extend across the plate from flange to flange. The enhanced stiffness would appear to prevent not only the buckling of the web but also the complimentary buckling of the adjacent flanges.

FE modelling of shear bond

The important observations regarding the mechanism of failure are confirmed by FE analysis and further conclusions are drawn as follows:-

- At the beginning of the load history, up to approximately 15 mm of midspan displacement, the mechanical interlocking in the shear span is the main contributor to the interaction resistance.
- Then, the mechanical interlocking is reduced as tensile strains increase in the sheeting and the friction at support becomes more important in anchoring the sheeting.
- In the last phase local buckling occurs causing a sudden drop of the slab resistance. The local buckling of the flange does not effect the strength but it effects the ductility of the considered slab, by reducing the yielding plateau.

Overall Conclusions

Experimental work on composite slabs and push-off tests, carried out in Strathclyde University, has been described. Several behavioural characteristics have been identified and their influence on strength and stiffness has been surmised. The data from the tests has been used in a Finite Element model prepared in the University of Technology in Lulea. Using the quantitative test observations it has been possible to make modifications to the program such that an excellent match occurs. This provides encouragement as to the future potential of the program to be used in slab analysis and design.

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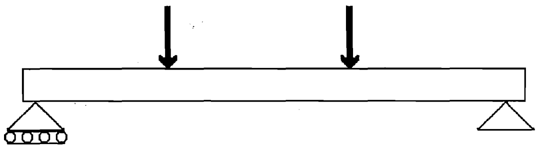
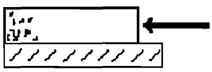
Table 1 Push Test details.

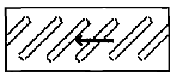
Profile	Embossed	Re-entrant portion	Ult load kN
Ward 60	no	yes	3.27
Ward 60	no	no	2.85
Ward 60	yes	yes	6.75
Ward 60	yes	no	5.97
Ribdeck 60	yes	yes	16.59
Ribdeck 60	yes	no	11.22
Holorib	yes	yes	7.73
Holorib	yes	no	4.88
Ward 80	yes	yes	7.8
Ward 80	yes	no	5.92



Macro:- Tests on complete systems



Meso:- Tests on components



Micro:- Tests on parts of components

Figure 1 Definitions of Macro, Meso and Micro level studies associated with composite slabs

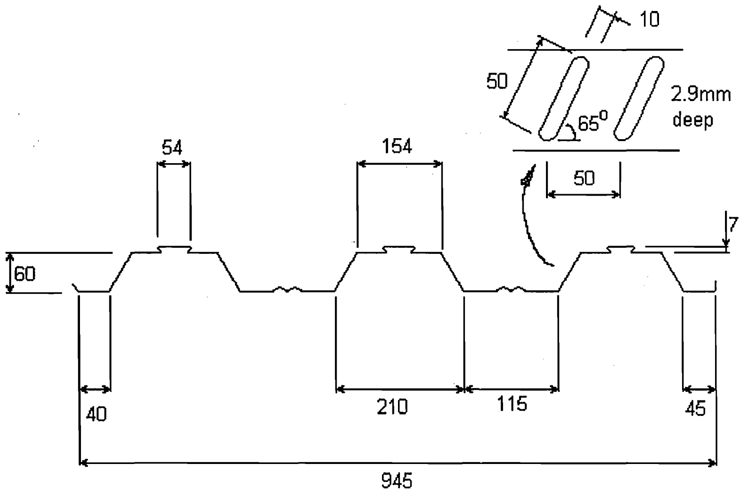


Figure 2 Ward 60 Profiled deck geometry

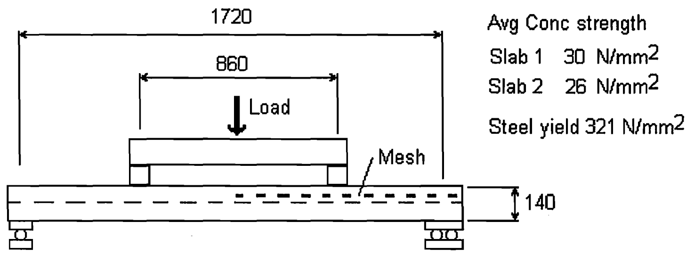


Figure 3. Composite slab test geometry

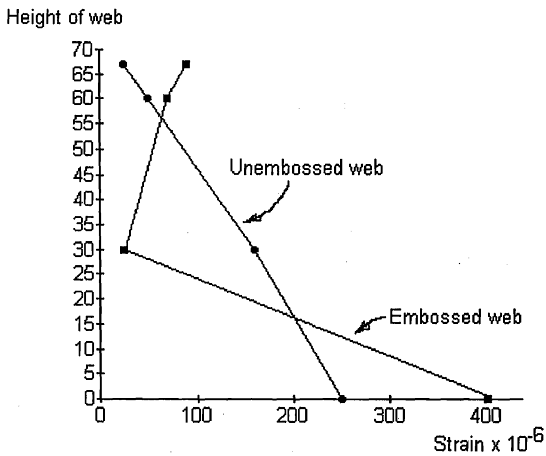


Figure 4. Strain in webs at an applied load of 40 kN

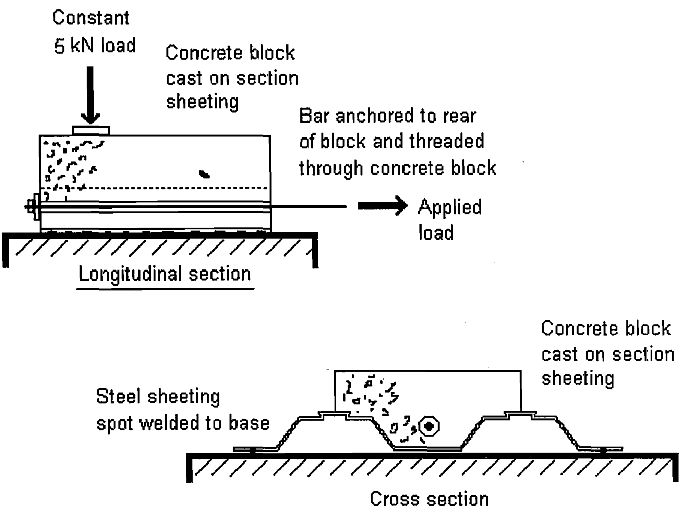
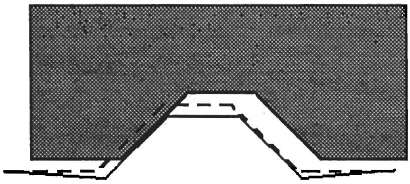
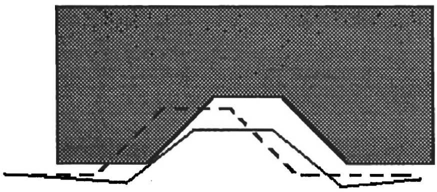


Figure 5 Layout of push test



Early loading where tension deformation is minimal



Later loading where tension deformation occurs

Figure 6. Deformation of the embossment due to tension

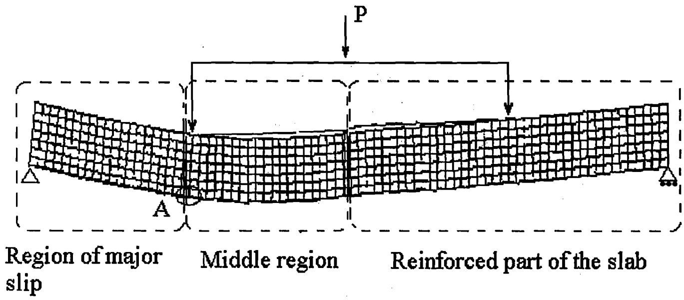


Figure 7 The FE model in deformed state

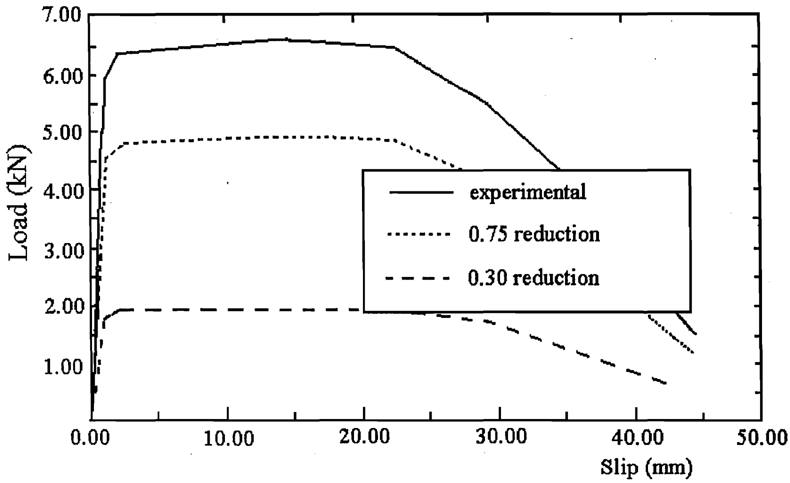


Figure 8. Mechanical interlock resistance

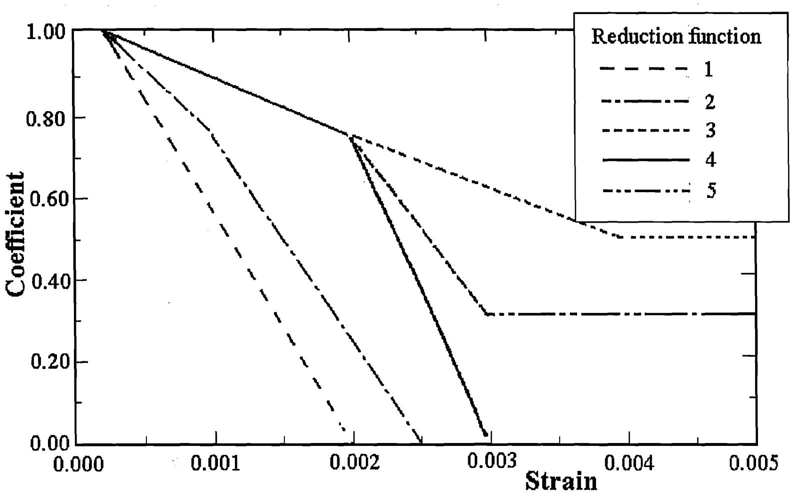


Figure 9 Reduction of mechanical interlock due to tensile strain

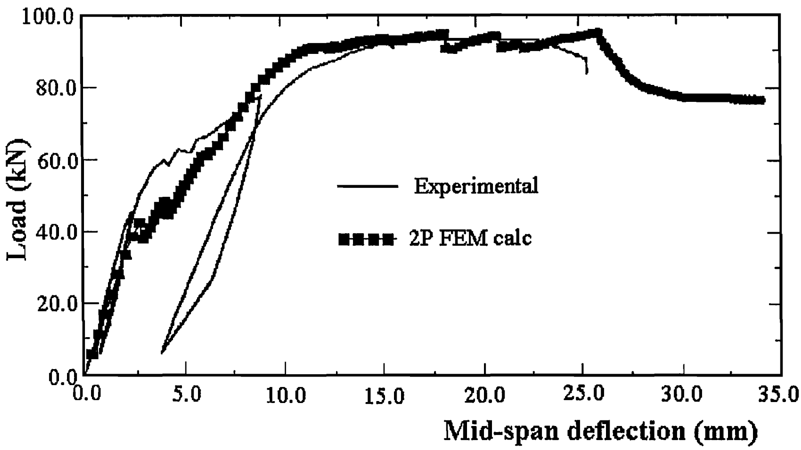


Figure 10 Load deflection for slab 2