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A Review of Composite Slab Design

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SUMMARY

64 composite slab tests are described and evaluated using 5 methods of analysis. One, a new plastic method, requires the imput of only one performance coefficient derived from relatively few tests. This offers significant economy over the current methods.

INTRODUCTION

Composite slabs are an efficient and economical method of providing flooring in steel framed buildings. Their popularity is largely due to the ease and speed with which the slab can be constructed and the efficiency of structural action developed.

The system is formed using profiled steel decking as permanent formwork and reinforcement to a concrete slab. The two materials act together as a composite system due, largely, to the action of shear embossments or keys pressed into the steel decking. The shear bond transferred between the steel deck and concrete depends upon parameters such as embossment size and depth, deck profile, steel sheet thickness and concrete grade and type.

Although it is possible for the keys or embossments to carry all the shear forces required to develop the full moment capacity of a slab it is more likely that a breakdown of shear bond will precipitate failure in most common deck profiles. The actual failure mode of the slab, in this case, is complex involving a shear failure in the concrete, local yield or buckling in the steel deck and excessive amounts of slip displacement between the concrete and steel deck (figure 1).

The complexity of the failure coupled with the number and variability of the parameters affecting the shear bond resistance of the deck has meant that purely analytical methods of predicting the ultimate load capacity of composite slabs have not been developed. Instead most methods of analysis rely upon performance coefficients that are derived from full scale slab tests specific to the deck under consideration. Consequently manufacturers are forced to carry out expensive performance tests on each deck profile in their product range.

A full scale test will give information on the failure load for a particular set of parameters such as span, depth, concrete grade, steel sheet thickness etc. The number of variable parameters may, therefore, define the number of tests required. A common situation occurs when a manufacturer of a particular deck wishes to evaluate the performance for variations in slab thickness, concrete strength and span length. A relationship between each parameter and the strength of the slab can be found from the tests. This indicates that three sets of three tests are needed giving information on the three parameters under consideration. (It is assumed here that at least three tests should be performed in each set to eliminate the possibility of rogue results affecting the outcome). In fact it is often the case that two or more parameters are interlinked or can be treated analytically and fewer tests will therefore be required.

Unfortunately the complexity of the failure mode and the interlinking of parameters means that the coefficients derived from the tests cannot then be definitely attributed to a particular parameter or failure mode. The analysis methods that use test derived coefficients are, therefore, rarely logically based on fundamental principles.

The authors have carried out 64 composite slab tests to determine 5 sets of design coefficients for various manufacturers. These tests have enabled a study of the behaviour of composite slabs of varying thickness and deck geometry, embossments of varying size, shape and depth and the suitability of various analytical methods in predicting slab strength.

This study is described in this paper and has led to an alternative method of analysis being proposed. This method is derived, in a logical and analytical way, from the failure mechanism observed. This, in turn, has led to the conclusion that only one test derived coefficient is necessary to reliably predict the behaviour of slabs. Many of the parameters thought to affect shear bond strength either do not do so, can be evaluated without the requirement of testing, or can be incorporated in a single test value.

EXPERIMENTAL WORK

Most of the tests carried out by the authors have been as a result of requests by manufacturers for load/span information for inclusion in brochures. The British Code of Practice for the design of composite slabs defines a test procedure from which two coefficients m_r and k_r can be determined. These coefficients are similar to those developed by Schuster and by Porter and Ekberg. The coefficients may then be used to determine loads and spans for slabs formed with the same deck but with various slab depths and concrete strengths.

The tests were all carried out on simple spans loaded with a symmetrical arrangement of either two or four line loads. The four line load arrangement approximated a uniformly distributed load which is considered to be the normal design situation for these slabs. Two line loads were only used for short slabs where four line loads would have proved difficult to arrange.

Most slabs were cast with a lightweight aggregate concrete and included a light mesh reinforcement just below the slab surface. In most cases the slabs were cast unpropped, that is they were supported only at each end and the deck was allowed to deform under the load of wet concrete. In addition thin steel sheet crack inducers were incuded at the inner load points. These ensured that the tensile capacity of the concrete would not beneficially affect the behaviour.

An initial dynamic load test was carried out during which the slab was cyclically loaded ten thousand times between one and a half times and a half of the assumed working load. A static load test to failure was then carried out.

According to the Code each deck type requires at least six and preferably eight tests for the determination of the coefficients. Half of these should be carried out on as short a slab span as possible and the remaining half on as long a span as possible. Consequently, a considerable number of slabs need to be cast and cured. The constraints of a busy laboratory led to many of the slabs being cast in the open at a field testing station some way from the university campus. A mobile purpose made trailer rig was constructed so that the load tests could also be carried out at the field station. This rig is shown in figure 2. and further details can be found in a paper by Wright and Peetham-Baran.

Although the British code recommends that six or eight tests are sufficient to determine the relevant coefficients several of the test series involved up to twelve slabs. The additional tests were commissioned to investigate extra long spans or very deep slabs. Several of the test series involved just three tests carried out to confirm the behaviour of a slab at a particular span or in a particular situation. Consequently, a much fuller picture of behaviour has been built up over the test period.

EXPERIMENTAL EVALUATION

This bank of test information has led the authors to several conclusions with regard to the behaviour of composite slabs. Some of these have already been recorded in previous papers. The major conclusions are, however, itemised below.

1) All of the slabs failed by loss of shear bond between the deck and concrete with a diagonal tension crack forming at approximately one quarter of the slab span. This occured in slabs loaded with two line loads and four line loads although in the latter case vertical cracks were also noted immediately below the outer load position.

2) Long thin slabs failed in a ductile manner with considerable slip occuring between the deck and concrete prior to failure. Short thick slabs tend to fail in a sudden brittle manner. This change in ductility between long and short slabs of the same deck type has been observed in two of the five test series.

3) Concrete strength does not appear to affect the strength of the slabs. Slabs with measured concrete strengths of only 2322 psi (16 N/mm^2) behaved in a similar way and gave similar ultimate loads to identical specimens with much higher concrete strengths. It is, however, prudent to assume that there is a lower bound to this observation!

4) The depth of the embossment or shear key is critical to the strength of the slab. Two series of tests were conducted on slabs with decks identical apart from embossment depth. It was found that a reduction in embossment protrusion from .098 in. (2.5mm) to 0.067 in. (1.7mm) caused a 66% reduction in load capacity.

5) Shallow decks have a tendency to separate and curl away from the concrete slab during testing. This reduces the observed strength of the slab in the test but may not be of importance in

practical situations where the breadth of the slab formed by several sheets side by side is effectively very wide.

6) The fact that slabs have been dynamically loaded before a static test to failure will only affect the ultimate load capacity of the slab if a critical amount of slip between deck and concrete has occured. Many of the slabs tested displayed end slip between the deck and concrete during the dynamic loading and in certain cases this slip increased progressively during this stage. If the slip increased to a value and then stabilised a much higher ultimate load could be expected during the static test. If, on the other hand, the slip was still increasing as the 10,000 cycles were completed only marginal increase in load capacity could be obtained.

These observations accord well with those of other researchers and it is believed that the qualitative behaviour of the slab at failure is now well established. Establishing a method of quantitatively predicting the ultimate load capacity of composite slabs has not been as successful.

DESIGN METHODS

As stated earlier each test series was carried out as a result of a request by a manufacturer for specific load span information. Most of the test series were, in fact, carried out in order to evaluate m_r and k_r coefficients as defined in the British Code of Practice. The two coefficients are obtained from tests with extreme slab spans and slab depths and can therefore be used to calculate the ultimate loads for the same deck and any intermediate slab span and slab depth and also, since concrete grade is included in the method, for any concrete grade.

It is difficult to assess the validity of the design method when the tests themselves have been used to obtain coefficients upon which the accuracy of the method depends. A suitable measure of whether the method is accurate is a value of standard deviation obtained in the following manner:-

1) The test results are used to evaluate the coefficients relating to the deck type.

2) These coefficients are used to evaluate the theoretical load capacity of each slab using this deck.

3) The test load capacity is expressed as a percentage of the theoretical load capacity for each slab.

4) The standard deviation of the values obtained in 3 is evaluated.

This standard deviation gives the likely error in percentage terms between test and theory.

Table 1 presents these standard deviation values for each of the test series. Values are given for several methods of analysis as well as the British Code method (denoted in the table as the m_r k_r method).

In two of the eight groups tests were carried out on only three slabs and it is realised that representative coefficients cannot be obtained with so few results. However, in test series two the deck was nominally identical to that used in series one apart from the fact that production rather than prototype specimens were used. In the case of series 5 the deck used was identical to the series 4 deck although the tests were carried out on specimens cast with a single temporary prop. The standard deviations recorded for these decks have been evaluated assuming the deck coefficients for the combined test series on the same deck type.

Each of the design methods will now be discussed in turn.

a) The m_r k_r method

The m_r k_r method can be seen to predict the ultimate load to a standard deviation of 16.4% in the worst case. This is quite large and some explanation regarding the scatter of results must be given. Test series three and six were carried out on specimens with considerable variation in slab thickness. As mentioned in the test observations thick slabs tend to fail in a more brittle and less predictable way. This is thought to be the reason for the large standard deviations recorded in test series 3 and 6. Deck 7 was a prototype deck that was formed by folding rather than rolling the steel sheet. In this deck no stiffeners were incorporated in the flanges and the deck flexed considerably during testing. For the remaining tests the standard deviation recorded is less than 5.2.

It can be concluded that the m_r k_r method will give acceptably accurate results when only the slab span is varied appreciably. It is also interesting to note that the inclusion of concrete grade in the method may affect the results. This is contrary to observation 3. The equation, upon which the method is based, is given below.

$$V = B\, d\, (\, m_r\, A\, /(B\, L_v) \; + k_r \;\; f_{cu})$$

where V is the shear resistance.
B is the breadth of the slab.
d is the effective depth of the slab.
A is the cross section area of the steel deck.
L_v is the shear span.
f_{cu} is the concrete crushing strength.
m_r and k_r are test derived coefficients.

It can be seen that increasing the concrete grade will affect the shear resistance V. If this is done for the short spans but not the long spans a higher value of m_r will result. Consequently a manufacturer who specifies high strength concrete for short span tests and weak concrete for long span tests will get a higher m_r coefficient. This is despite the fact that several authors have shown that concrete strength has no influence on the load capacity of the slab.

b) The Seliem Shuster method.

The British code does not make specific reference to the steel sheet thickness and it is has been assumed that the m_r k_r coefficients determined from the performance tests are valid for any thickness. This may not be the case and American and Canadian codes require decks to be tested separately even though the only parameter variation may be steel sheet thickness. Consequently a manufacturer who uses the same roll former for several steel sheet thicknesses will be required to carry out seperate sets of tests for each thickness even though the geometry of the deck will otherwise be identical.

Seliem and Shuster addressed this problem and proposed the design formula given below.

$$V = B\ d\ (\ (k_1\ t\ /\ L_v) + (k_2\ /\ L_v) + k_3\ t + k_4$$

where V, B, d and Lv have the same meaning as before
t is the thickness of the steel sheeting.
k_1, k_2, k_3 and k_4 are test derived coefficients.

Each of the factors k_1 to k_4 have to be determined from a multi-linear regression analysis. It can be seen from this formula that concrete grade is omitted with slab thickness, slab span and steel sheet thickness being the variable parameters. This significantly reduces the number of tests required for many manufacturers product ranges.

The fourth column of table 1 shows the standard deviations, obtained in the same way as before, for the slabs tested and analysed using the Seliem Shuster formula. A very similar pattern of results to those obtained using the m_r k_r method can be seen. This is to be expected as most of the test series were carried out on decks of one thickness.

Test series six was, however, carried out with three tests with a steel sheet thickness of 0.9mm (approximately equal to 20 gauge) and the remaining tests with a steel sheet thickness of 1.2mm (approximately equal to 18 gauge). The Seliem and Shuster method does appear to be more accurate than the m_r k_r method for this deck.

Test series two was carried out on only three specimens with decks of 0.9mm (approximately equal to gauge 20) steel thickness and could, therefore, not be used to evaluate the performance coefficients. The actual coefficients used were obtained in test series one, the deck for which was identical apart from being a prototype of 1.2mm (approximately equal to gauge 18) steel thickness. If the Seliem Shuster method is of better accuracy then the standard deviations recorded in test series one and two should be similar. This is clearly not the case and this would indicate that the method is inaccurate although in this case the variation may be due in part to the difference between prototype and production decks.

c) Prasannan and Luttrell method

In both the previous methods tests on sample decks are required to establish empirical coefficients that are then used to evaluate slab performance in the general case. The tests are expensive and a design method that reduces the requirement of testing has long been the aim of researchers. As stated previously the complexity of the parameters effecting the behaviour means that a purely analytical solution is some way off. However Prasannan and Luttrell have suggested an alternative.

From a considerable number of tests on many deck types they identified trends in behaviour associated with a number of parameters. They were then able to produce empirical coefficients that, based on a very large test sample, were able to predict the performance of decks without the need for testing.

The method proposed by Prasánnan and Luttrell is unusual in that, rather than evaluating coefficients that modify the shear capacity of the slab, they devised relaxation coefficients which were applied to the ultimate moment of resistance. The basic formula is presented below.

$$M_t = (k_3 / (k_1 + k_2)) M_f - k_4$$

where M_t is the moment capacity of the slab.
M_f is the moment capacity of the slab based on a full plastic section.
k_1, k_2, k_3 and k_4 are empirical coefficients.

This formula can be seen to give a moment capacity as a proportion of the maximum moment capacity of the slab. The factors k_1 to k_3 relate to the properties of the deck; factor k_1 is dependent upon the deck geometry, factor k_2 on the steel sheet thickness and slab depth and k_3 on the width of slab and pitch of the profiling. Factor k_4 is dependent upon the shear span of the slab.

Each of these factors were evaluated by Prasannan and Luttrell from the considerable test data available to them and have been presented in empirical equations and design graphs. It was therefore possible to apply these factors to the tests recorded here.

As the load capacity of the test slabs can be evaluated directly a comparison based on standard deviations is misleading. The variation of load capacities calculated may be small but the mean load capacity may be substantially different from the test value. This is shown clearly in column 5 of Table 1. Column 5 shows the percentage difference between the mean test result and mean result computed from the Prasannan Luttrell method in brackets after the standard deviation value. This can vary by as much as 57.4% and clearly indicates that the method is highly inaccurate.

Two conclusions may be drawn from these observations. Firstly the coefficients derived from American testing by Prasannan and Luttrell do not appear to describe the behaviour of the deck profiles tested by the authors. It is, however, possible that with different coefficients the mean of the computed values could be much closer to the mean of the test results. Secondly as the computed standard deviations are high the formula itself would appear to be inaccurate.

Although this is a severe critisism of the method the authors believe that the approach used is worthy of further work. It may well be possible to correlate the test results of a large number of tests and derive improved coefficients that will predict behaviour well. However the base formula used to describe the coefficients is critical and a version based upon shear bond resistance rather than moment capacity may be more suitable. Once an analytically sound formula has been derived it will then be possible to isolate particular parameter variations and ascribe particular coefficients.

d) The Partial Interaction Method

This method has been proposed by Bode and Stork as an alternative to the m_r k_r method for the draft E.C.4. code of practice. The method is based upon the development of a plastic stress block at the maximum moment position along the slab span. It is assumed that this may be the full plastic moment capacity of the section if sufficient connection is provided between the load point and the support. In most cases the shear bond capacity of the embossments or keys is not sufficient for this to occur and partial connection results. If no connection is provided then the moment capacity is only the plastic moment capacity of the steel deck alone.

Figure 3 shows the relationship between the minimum and maximum moment capacity for a typical slab. The curve shown can be approximated as a straight line and, as the moment capacity of the sheeting alone is often very small, it is sufficiently accurate to assume that the line will pass through the origin.

For any slab type it is necessary to carry out tests to determine the slope of the line. As only one coefficient, the slope of the line, is neccessary very few tests are required. In practice manufacturers would probably wish to carry out more tests and use a more accurate curve based on the actual stress block rather than the straight line.

Once again the authors have back analysed the slabs tested by them using this method. In this case the straight line approximation passing through the point on the vertical axis equivalent to the moment capacity of the steel deck has been used. It can be seen from column 6 of Table 1 that the standard deviations recorded range to a maximum value of 27.5. There is some consistency with the m_r k_r method although the numerical values of the deviations are higher and this indicates that the method is less accurate.

The method does, however, hold two advantages over the other methods discussed so far. Firstly it is a logical derivation from observed structural behaviour that involves a meaningfull coefficient i.e. the degree of interaction. Secondly it is possible that relatively few tests will be required to determine the degree of interaction for a particular profile.

Each of the methods described above has been developed to reduce the requirements for performance testing. It is clear from the comparisons between the methods that many hold little advantage over the original m_r k_r method. It is only the last of the methods, the partial interaction method, that offers a reduced number of tests, a logical analytical base and a test derived performance coefficient that has physical meaning. Unfortunately the accuracy is less than that of the m_r k_r method and the coefficient derived in the tests can only apply to identical deck types.

DEVELOPMENT OF A NEW PLASTIC COLLAPSE METHOD

The authors have also developed a new method based upon their experimental observations. The main observation has been the ductile collapse mechanism of most of the slabs. This indicates that a plastic method of analysis is a suitable alternative to the more normal equilibrium method.

It has already been stated that all the slabs failed in a shear bond mode with a diagonal tension crack forming at a position defined by the shear span and horizontal slip occuring between the steel sheet and the concrete. The slab collapses as a hinge forms at the shear span as shown in figure 4. It is interesting to note that in the tests the plastic hinge always occured at the quarter span for slabs loaded with either the two point or four point load system.

The authors reason that the energy applied to the slab in deforming the hinge must be equal to the energy resisting the rotation of the hinge. The external energy can easily be calculated as the applied load multiplied by the distance through which it moves. This is shown in figure 4 for both a two point and four point load system. The energy resisting the rotation of the hinge has two components: the plastic moment capacity of the steel deck multiplied by the rotation of the hinge and the force resisted by the shear keys multiplied by the distance over which the concrete has to move in relation to the deck (the slip). Figure 5 shows diagramatically the energy resisting the rotation of the hinge.

The externally applied energy and the internally resisted energy can be equated and for the simple case of a slab subject to a two point load gives the following expression:-

$$(W\ L)\ /\ 8 = m + F\ d$$

where W is the applied load.
L is the slab span.
m is the moment capacity of the steel deck.
F is the connection force.
d is the effective depth.

The connection force, can be assumed to act over the surface area of the steel deck and it is therefore possible to evaluate a value of shear bond for each deck type.

$$f_{sb} = F\ /\ (L_v\ B_{full})$$

where f_{sb} is the shear bond
B_{full} is the total breadth of steel sheet.
L_v is the shear span.

For the case of a four point load the external energy needed to create a unit deformation of the hinge position can be seen, from figure 4, to be less than in the case of the two point load. In addition the shear bond is unlikely to develop uniformly over the entire shear span but will be highest over that eighth of the span length, closest to the support, which has the highest vertical shear load as shown in figure 6. This has been confirmed in tests where small vertical cracks are found immediately under the outer load point in the four point load system (see figure 7). Had the whole quarter span length been carrying vertical shear this crack is unlikely to have occured.

This gives rise to the following relationships between load and shear bond in each of the two cases.

For a two point load:-

$$(W L) / 8 = m + f_{sb} (L_v B_{full})$$

For a four point load case:-

$$(3 W L) / 32 = m + f_{sb} (L_v B_{full} / 2)$$

where all terms are as defined before.

These expressions have been used to back analyse the 64 slab tests and standard deviations, evaluated as before, are shown in column 7 of Table 1. These results are comparable with the m_r k_r method with generally only marginal loss of accuracy. One particular anomolous result, that for test series 7, deserves comment. This test series was carried out on prototype decks with no longitudinal rib stiffeners and considerable curling of the edges of the deck occurred during the tests. This is thought to have influenced the shear bond between steel and concrete more for long spans than short spans and has given rise to a consequent discrepancy.

The plastic method has some similarity with the partial interaction method. The test results may be used to evaluate either a degree of interaction in the case of the partial interaction method or a value of shear bond in the case of the plastic method. This value may then be used to evaluate moment capacity at the position of highest moment which, in the case of the plastic method, is assumed to be at the shear span.

In both cases, only one coefficient is required and it is therefore acceptable to carry out only three tests for its evaluation. This fact has been confirmed by evaluating a coefficient for each of the decks using only three of the tests. The degree of interaction and the value of shear bond was found to be only marginally different from the mean value derived from all of the tests in each series.

CONCLUSIONS

The authors have carried out 64 composite slab tests and have been able to identify the characteristic behaviour of the system. Five separate methods of analysis have been used to evaluate the performance of the decks and the results have been compared. One of the analyses has been developed by the authors and provides a simple and accurate method of evaluating the performance of composite decks using a minimum number of qualification tests. Detailed conclusions are listed below:-

1) Composite slabs normally fail as a result of a critical loss of shear bond between the steel decking and concrete. This manifests itself by a diagonal crack at a position known as the shear span and interface slip between the steel and concrete.

2) Most slabs fail in a ductile and predictable way with around 2mm of measured slip occurring before failure. The concrete strength does not appear to affect the slab strength as long as it is over a certain minimum value. Embossment protrusion has a significant effect on slab strength.

3) Most methods of predicting slab strength depend upon factors evaluated from performance tests. The number of tests will vary depending upon the number of variable parameters included. The most popular method known as the m_r k_r method in Britain requires a minimum of six tests for each profile type.

4) Several methods of analysis have been developed in order to try and reduce the reliance on test information. Generally these offer no benefit in terms of accuracy and may involve considerable computational effort.

5) A plastic method developed by the authors follows a simple logical failure mechanism and gives equivalent results to the m_r k_r method with only three rather than six tests. This may offer a significant saving on the costs of preparing load span tables.

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APPENDIX 2 NOTATION

A	Cross sectional area of the steel sheeting.
B	Breadth of slab.
B_{full}	Full width of steel in sheeting.
d	Effective depth of slab.
F	Connection force.
f_{cu}	Concrete crushing strength.
f_{sb}	Shear bond.
$k_1 - k_4$	Coefficients.
k_r	Test derived coefficient.
L	Slab span.
L_v	Shear span.
M_f	Moment capacity of full plastic section.
M_t	Moment capacity of slab.
m	Moment capacity of steel deck.
m_r	Test derived coefficient.
t	Steel sheet thickness.
V	Shear resistance.
W	Load on slab.

Table 1

Comparison of Composite Slab Analyses

Test Series	No. of slabs	mr kr	Seliem Shuster	Parasannan Luttrell	Partial Interaction	Plastic Method
1	10	5.2	4.9	14.1(1.6)	6.9	5.4
2	3	13.2	10.4	15.6(23.7)	10.3	9.3
3	10	16.4	17.2	13.7(-24.3)	19.4	15.2
4	6	4.8	3.7	42.6(18.8)	8.6	6.5
5	3	11.4	9.9	20.0(47.5)	11.1	15.3
6	12	12.6	7.8	12.5(38.7)	20.4	6.3
7	12	15.3	15.6	12.6(-27.0)	27.5	22.2
8	8	3.2	3.4	24.8(57.4)	13.3	5.3

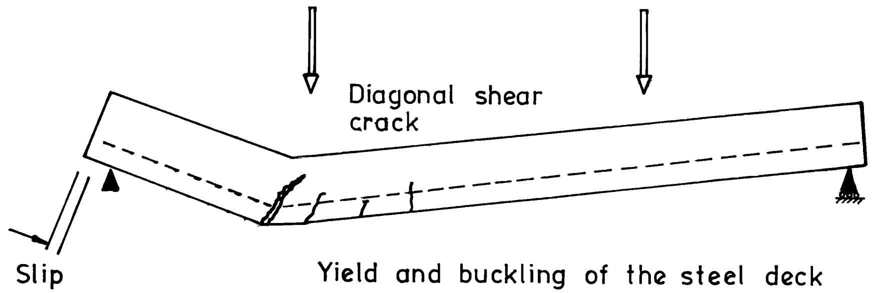

Figure 1 Composite deck failure

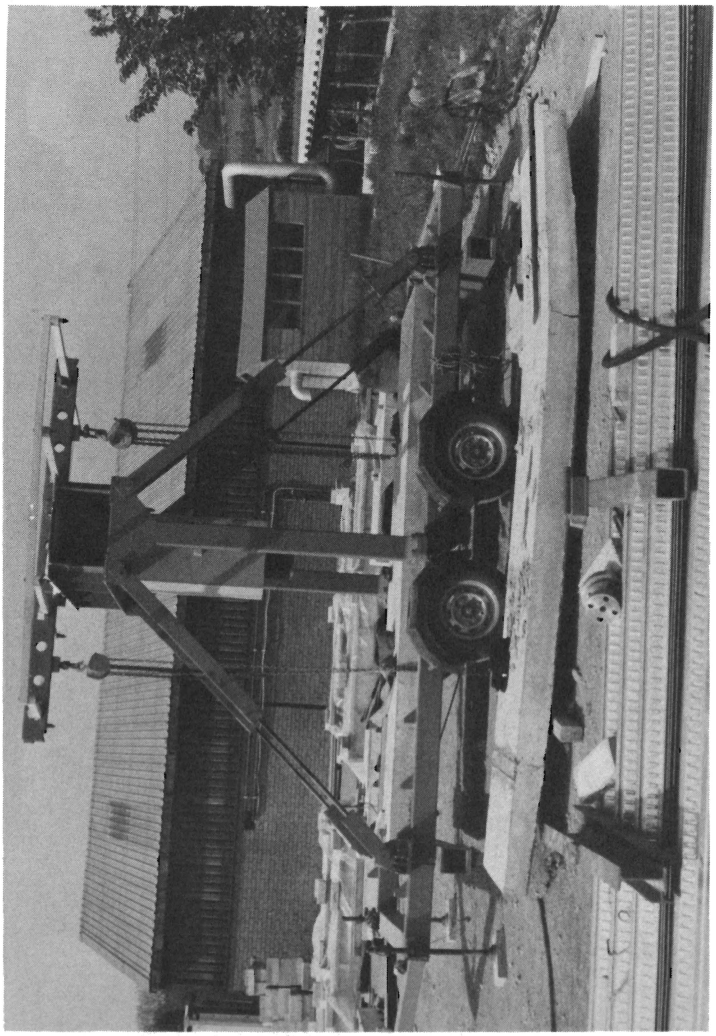

Figure 2 Trailer rig

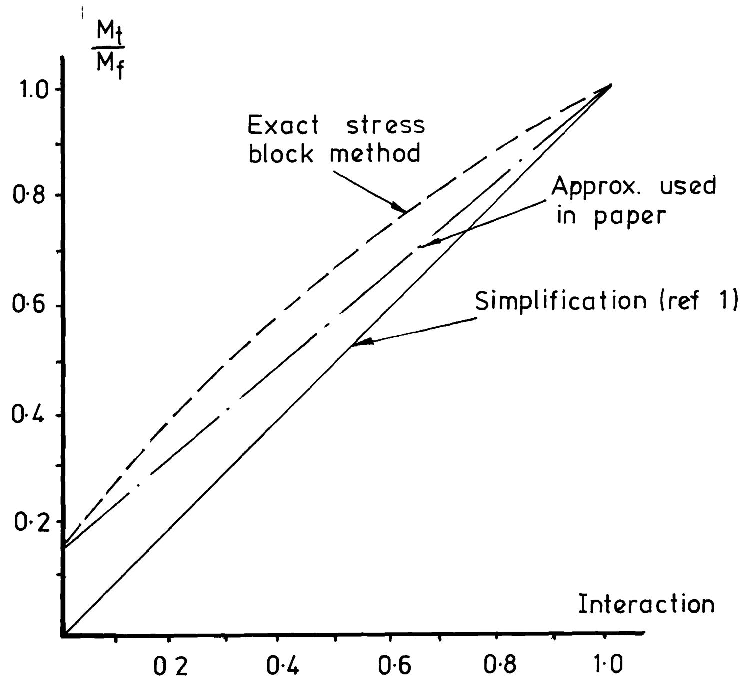

Figure 3 Partial interaction

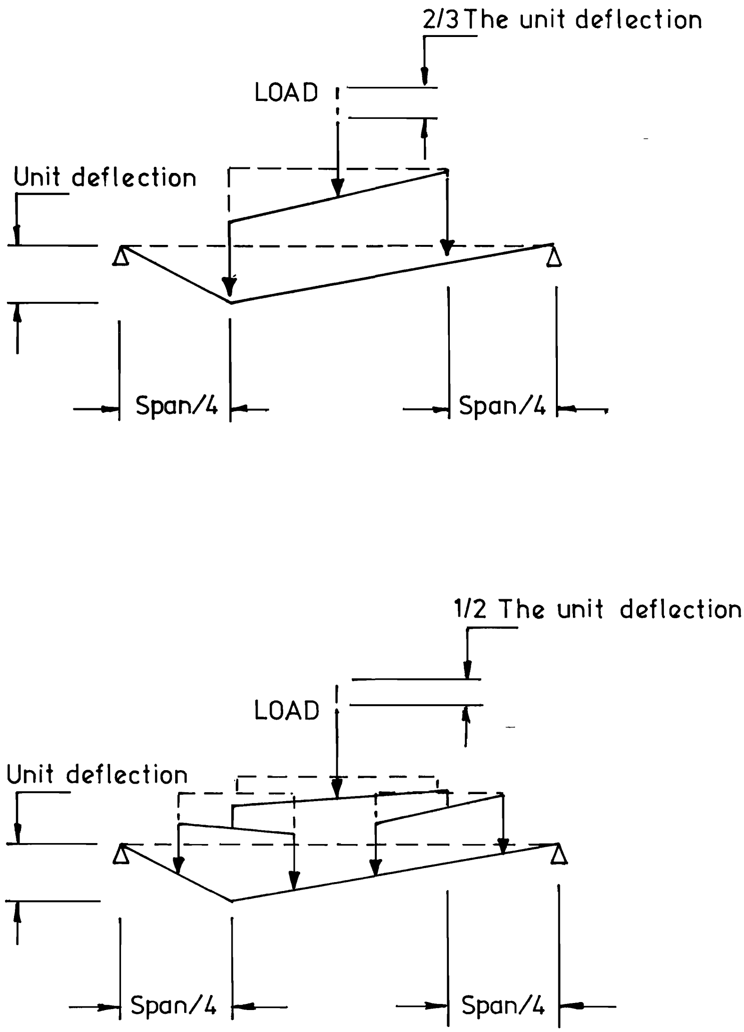

Figure 4 External load movement

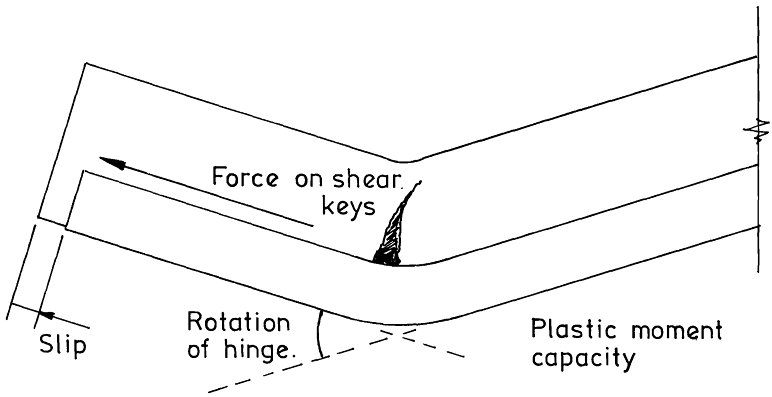

Figure 5 Internal energy

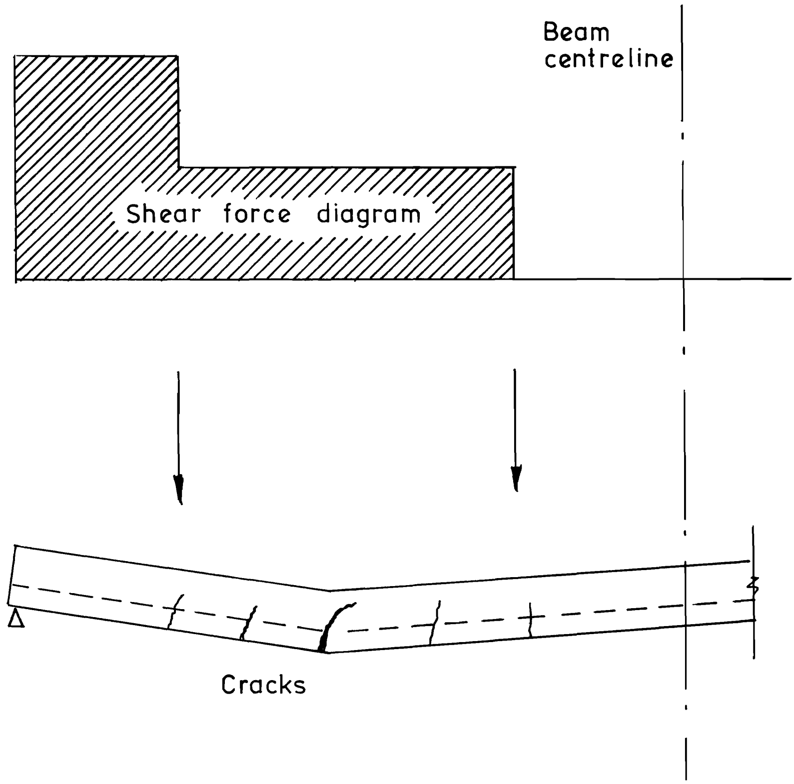

Figure 6 Vertical shear forces

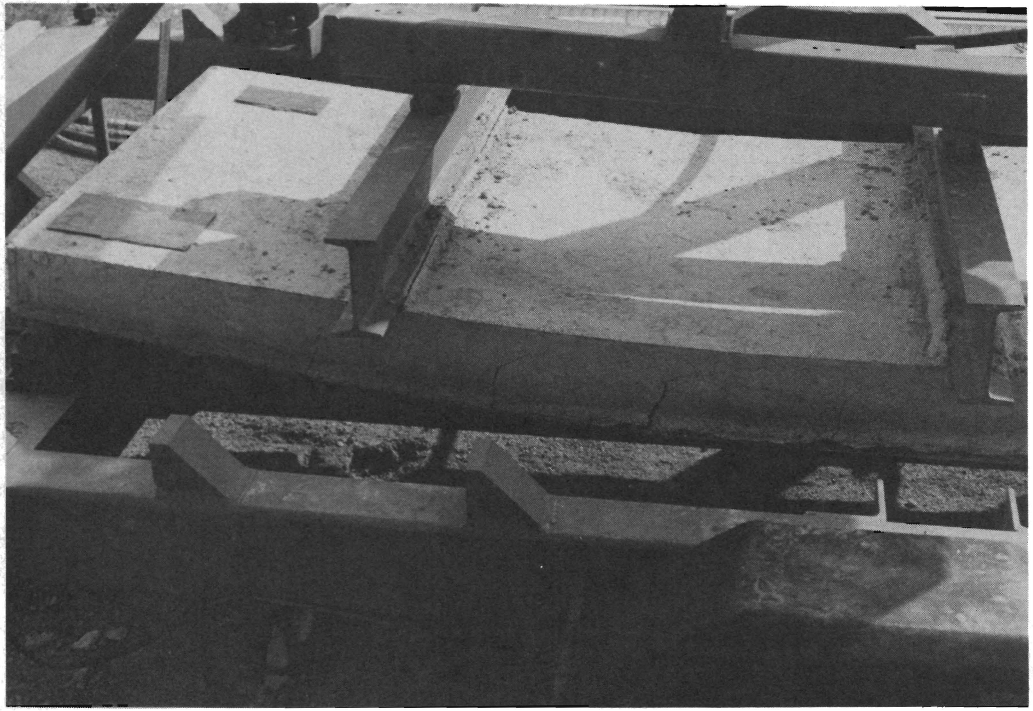

Figure 7   4 line load test